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REPORTS, PAPERS, DISCUSSIONS, AND MEMOIRS

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AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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THE VIRGINIAN RAILWAY ELECTRIFICATION

By George Gibbs,* M. Am. Soc. C. E.

To Be Presented February 1, 1928

Synopsis

The paper aims to present the broad engineering features of a recent and important electric traction installation for long-distance freight service of the heaviest character, in a manner which will make clear to railway engineers generally its essentials in design, construction, and operation. In so doing an attempt has been made to avoid undue length or complication in describing details which are of interest to electrical engineers only.

The physical character and equipment of the Virginian Railway on its heavy grade division, the character and amount of business, and the difficulties experienced in caring for its rapid growth under steam conditions, are first described. The paper then proceeds to outline the methods followed in determining quantities and types of apparatus required to handle the given traffic electrically. A general description of the main elements of an electric traction system is given, and the characteristics of the particular system determined upon as most suitable for the given conditions are then briefly described under the usual sub-divisions of the power supply, the high-tension transmission line, the distribution devices along the railway, and the locomotives.

In this treatment stress is laid on some of the special problems solved, such as the adaptation of the combustion features of the power plant to handle successfully the very large momentary fluctuations in power demand required to start the heavy trains on a grade. Also, the inductive effects set up in paralleling electric circuits normally and during short-circuits are explained, as well as the corrective means adopted. The problems involved in the design of a suitable continuous-contact wire system over the tracks are described

Note.—This paper is issued before the date set for presentation and discussion. Correspondence is invited and may be sent by mail to the Secretary. Discussion on the paper will be closed in August, 1928.

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fully, and the requirements for a successful catenary system and its supports are illustrated, as matters which are vital to a successful traction system and which present features affecting the work of railway engineers generally.

A short description is given of the inspection and repair facilities provided for the locomotives. Also, the special features necessary for the Company's telephone and telegraph lines along the tracks are noted.

The paper closes with a short description of the manner of conducting the designing of the system throughout, the special methods followed in its construction, and a résumé of operating experience.

Introduction

Within the western slopes of the Appalachians in West Virginia is located a series of bituminous coal deposits of great extent, notably the "Pocahontas", the "New River", and the "Kanawha" fields. Because of the favorable mining conditions, and especially because of the high calorific value and free-burning qualities of the coals from these fields, they are in increasing demand for steaming purposes. They are made available in the markets, both East and West, by three railway systems, the Chesapeake and Ohio, the Norfolk and Western, and The Virginian. Conditions surrounding transportation to the Atlantic seaboard only are, however, dealt with in this paper. These three railways have their tide-water termini at ports around Hampton Roads, Virginia, to which, in 1926, they transported more than 27 000 000 gross tons of coal for vessel loading. Of this amount, 45% was "export" coal, that is, destined to points outside the United States, and the remainder was for domestic consumption. Of the eastbound coal, more than 75% was classed as "low volatile".

The Allegheny Mountains present a complicated topography, characterized by a series of lateral spurs having steep contours and enclosing narrow valleys. Under these conditions favorable railway location is difficult and long adverse grades eastbound are necessary, with many viaducts and tunnels and a large amount of curvature. Furthermore, it is extremely expensive to provide additional running tracks; and to secure room for passing sidings is always a problem, in places, impossible. Unfortunately, also, these difficult conditions obtain in a section where cars must be distributed to and gathered from mine sidings and spurs.

To compete effectively with Pennsylvania mines, which are at least 100 miles nearer the seaboard and, perhaps, several hundred miles nearer the great coal-consuming centers, these more distant West Virginian coal fields must enjoy relatively low mining cost and be served by very efficiently operated railways to secure minimum transportation costs. One important factor contributing to cheap transportation is the maintenance of an intensive service with trains of the maximum permissible weight moved at a speed which is as high as safety permits. Thus, by quick transference of lading from the cars to vessels at tide-water, the completion of a prompt "shuttle service" between mines and terminal becomes possible. This character of "turn-around"

obviously greatly reduces the equipment required to perform a given service and thus contributes in an important way to the possibility of profit from the low freight rate prevailing in this highly competitive commodity.

Electric Traction on the Norfolk and Western Railway

There is an especially close similarity between the Norfolk and Western and the Virginian Railways, as to grades and alignment in the coal fields and as far east as Roanoke, Va. (Fig. 1). Each has a "gathering section" west of the mountains and must haul eastbound tonnage over heavy grades to the summit before favorable line conditions are reached. In fact, the two roads lie closely parallel for almost the entire distance from the coal fields to tide-water. The experience of the Norfolk and Western Railway Company, which had built up a large eastbound coal business before the Virginian Railway was constructed, may therefore be briefly referred to as indicating modern methods in the development of economical mass transportation of low-grade freight, especially as bearing upon the advantages of electrification to this end.

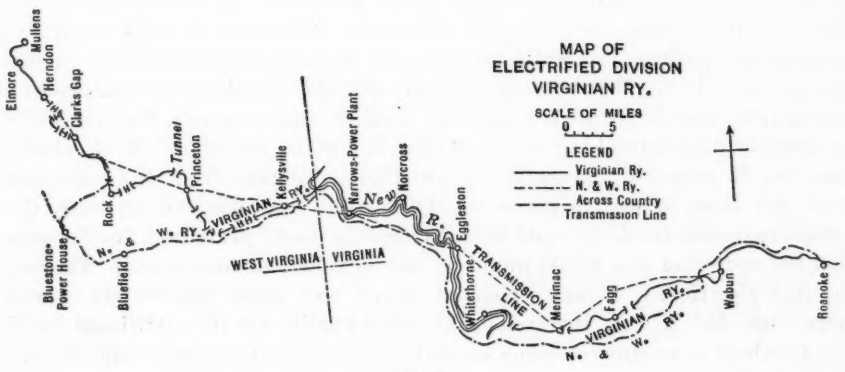


Fig. 1.

The Norfolk and Western Railway Company had used powerful Mallet compound steam locomotives to haul very heavy trains on the mountain grades. The service speed was low, and three locomotives were required per train. In the coal-"gathering" section, comprised within about 30 miles of road immediately west of the mountain summit, the line was double-tracked (except in the single-track ¾-mile Elkhorn Tunnel). With these facilities, however, it was found that service had about reached its maximum possible capacity and economical development. Additional main tracks could only be provided at prohibitive cost through the narrow valleys, where the road was encumbered with mine sidings and coal tipples, and the number of steam locomotives required on this short section was so great as to be a positive hindrance to prompt movements. For example, to handle the coal tonnage of 1914, twenty-four Mallet locomotives were required in as many miles of line, and a round trip—say, 50 miles per day—was about the best average performance which could be secured from each.

The electrification of this "gathering" section of the railway was adopted after an extended investigation as the best solution of operating difficulties and limitations present and future. The installation was completed and put into operation in May, 1915. The new traction method at once effected a reduction in the number of locomotives in service to about one-third that required by steam traction; it doubled, and at times trebled, the daily mileage per locomotive by increasing train speed and by eliminating delays inherent with steam locomotives. Thus, congestion was completely eliminated. During the World War business increased suddenly and very greatly, but notwithstanding this, traffic was handled electrically without difficulty and undue congestion, a feat which the operating officials believe would have been impossible with steam traction.

The Virginian Railway Problem

The Virginian Railway Company had recently to face similar problems; business had grown constantly until it reached a volume approximating that of its rival, which had more ample track facilities. A comprehensive plan for betterment, therefore, became imperative. In order to place itself in a comfortable position to handle by steam an anticipated progressively increasing business, it would have been necessary not only to add to its equipment of locomotives and facilities for housing, coaling, and watering, but, especially, to provide double-tracking of its entire Mountain Division. With electric traction, it appeared reasonable to anticipate, judging from the experience with this form of motive power on the Norfolk and Western Railway, that extensive double-tracking could be postponed for many years and, furthermore, that the operation as a whole would be very flexible and economical. To equip the line electrically, it was estimated, would cost about $15 000 000—a very large sum—but the net investment, allowing credits for the additional facilities required to secure the same capacity with steam traction, would be only about $5 000 000.

The important direct savings produced by electric haulage over steam are in fuel, time of train crews, and the better utilization of car equipment due to increased speeds, hauling capacity, and reduction in delays. Where, therefore, as in this case, the excess capital fixed charges are not too great, it is reasonable to expect that the improved form of motive power will be directly profitable from the savings and indirectly advisable by reason of its superior flexibility and reliability. After 1½ years of experience with electric haulage on this road, it is gratifying to be able to state that the expectations of the Company have been more than realized in all respects. The direct savings produced have more than met the charges on the net excess investment and the improved operating results have been marked.

Description of the Property.—The Virginian Railway extends from Deepwater, W. Va., at the head of navigation on the Kanawha River, to Sewalls Point, on Hampton Roads, a distance of 443 miles. It was constructed in 1905-07, with a view to obtaining the shortest route and the best possible grades and alignment from the coal fields to tide-water. At present, the average length

of haul from mines to tide-water is 405 miles. The road traverses the Pocahontas, Winding Gulf, and New River Districts of the semi-bituminous (low volatile, or "smokeless") fields and taps a portion of the Kanawha (high volatile) District. It is now serving a total of 110 mines, having a combined potential annual capacity of at least 18 000 000 tons and an estimated recoverable reserve in excess of 1 000 000 000 tons. An additional area having a potential capacity far greater than this figure is tributary to the road.

The Company has two large, modern coal piers at Sewalls Point, designed for the loading of any type of vessel, and each of the two will accommodate four large, or six small, vessels at one time. Each pier is equipped with car-dumpers which can unload a total of 210 cars, or 12 600 tons of coal, per hour.

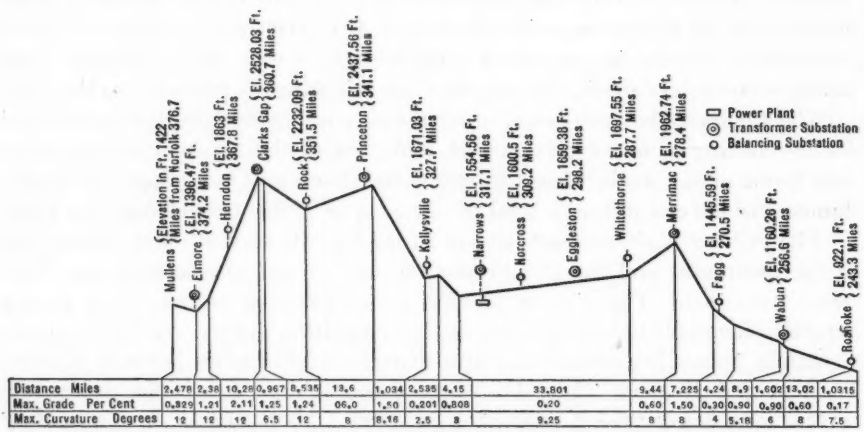
The railway is single track, except in the 22-mile section on the west slope of the mountains and the 10-mile section west of and approaching the Tide-water Terminal. The aim to provide a coal-carrying railway with grades generally favorable to loaded traffic has been attained, except over the two main mountain ridges intersected within a distance of 134 miles between Elmore, W. Va., and Roanoke (Fig. 1), where topographical conditions necessitate several long heavy grades, producing in effect a "bottle-neck" in the hauling capacity of the entire system.

This section now furnishes an example of the heaviest electric traction development to be found anywhere in the world. This project was undertaken, primarily, in order to equalize the capacity of the road throughout so as best to conform to the topographical peculiarities of the country traversed; but, furthermore, to avoid large expenditures for steam equipment, which, it was felt, would become obsolete within a short time. In this paper is given a necessarily brief account of this important installation, which under the local conditions, physical and operating, presented many new and difficult technical problems.

In order to keep the length of the paper within bounds, a full technical description of any of its features cannot be given. It was thought, on the whole, more useful to railway men and engineers to furnish a general description of the physical property and operating conditions, with sufficient explanation of the reasons, either operating or technical, for the adoption of the plans and methods followed, as a guide in determining the advisability of heavy electric traction for similar situations elsewhere.

Grades.—The alignment and profile of the Division are shown in Figs. 1 and 2 beginning at the west end of the line, in the Assembly Yard at Elmore, which is 1 400 ft. above tide-water. At this point the climb over the main mountain ridge starts and continues for 12½ miles on an average 2% compensated grade to an elevation of 2 528 ft. at Clark's Gap, W. Va.; thence over 22 miles of undulating and 0.6% maximum grade to Princeton, W. Va. Near this point the descent occurs and continues for 10.3 miles down a 1.5% grade to Kellysville, W. Va. Thence the line continues over a broken down grade for 6 miles into the valley of the New River, which is followed up stream for 34 miles over a 0.2% adverse grade to Whitethorne, Va. Here, the climb over the Allegheny Ridge begins. This requires about 10 miles of 0.6% grade; it was made a pusher section during steam operation. The mountain summit is reached at

Merrimac, Va., from which point the road descends sharply for 7 miles on a 1.5% grade. The remaining distance, 27 miles, to Roanoke is over light grades favorable to loaded movements.


Mullens {Elevation in Ft. 1422 / Miles from Norfolk 376.7
Elmore {El. 1396.47 Ft. / 374.2 Miles
Herndon {El. 1863 Ft. / 367.8 Miles
Clarks Gap {El. 2528.03 Ft. / 360.7 Miles
Rock {El. 2232.09 Ft. / 351.5 Miles
Princeton {El. 2437.56 Ft. / 341.1 Miles
Kellysville {El. 1671.03 Ft. / 327.7 Miles
Narrows {El. 1554.56 Ft. / 317.1 Miles
Norcross {El. 1600.5 Ft. / 309.2 Miles
Eggleston {El. 1659.38 Ft. / 298.2 Miles
Whitethorne {El. 1697.55 Ft. / 287.7 Miles
Merrimac {El. 1962.74 Ft. / 278.4 Miles
Fagg {El. 1445.59 Ft. / 270.5 Miles
Wabun {El. 1160.26 Ft. / 256.6 Miles
Roanoke {El. 922.1 Ft. / 243.3 Miles
Power Plant
Transformer Substation
Balancing Substation

Distance Miles	2.478	2.38	10.28	0.967	8.535	13.6	1.034	2.535	4.15	33.801	9.44	7.225	4.24	8.9	1.602	13.02	1.0315
Max. Grade Per Cent	0.329	1.21	2.11	1.25	1.24	06.0	1.50	0.201	0.808	0.20	0.60	1.50	0.30	0.90	0.90	0.60	0.17
Max. Curvature Degrees	12	12	12	6.5	12	8	8.16	2.5	8	9.25	8	8	4	5.16	6	8	7.5

FIG. 2.—TRACK PROFILE OF ELECTRIFIED DIVISION, VIRGINIAN RAILWAY.

Curvature.—Throughout the entire 134 miles there is a large amount of curvature, varying from 12° maximum and quite generally as sharp as 8°, with the worst curves on the heaviest grades; the mountain section west of Princeton has 60% of its length on such curves. It is evident from Figs. 1 and 2 that both grades and curvature have an important bearing on operating speeds and train loading, and it will be interesting, therefore, to outline the maximum possible service under steam operation.

STEAM HAULAGE METHODS

At Elmore, the main assembling point for eastbound tonnage, a train was made up of 5 600 tons trailing weight, summer loading. This was hauled over the heavy grade to Clarks Gap by a Mallet-type road engine and two Mallet pushers. At that point the pushers were cut off and returned light to Elmore Yard, the train continuing to Princeton with the road engine only. At Princeton, the train was filled to 8 500 tons, summer loading, or 6 000 tons, in winter, for the run to Roanoke. The power for this latter movement was one U. S. Mallet-type road engine, with a helper engine to assist starting out of Princeton Yard and a helper on the 10-mile grade from Whitethorne to Merrimac.

The average inclusive speed from Elmore to Princeton, with two stops for water, was about 7 miles per hour. Under best conditions 3½ hours were required for this 36-mile movement. This time, however, was generally exceeded and was often as much as 7 hours, due to bad weather or to poor steaming of the locomotives. The time required for the movement from Princeton to Roanoke was normarly from 11 to 12 hours at an inclusive speed of about 8 miles per hour; over-time was frequently required, however. In this

portion of the run stops were made at four stations for water, at one of which re-coaling and inspection took place.

Westbound the service was wholly an empty-car movement. The trains averaged 2 700 tons and were hauled over the Division by one road engine, without using a pusher on the Merrimac and Kellysville hills. The running time for this light service varied from 12 to 20 hours, depending on the weather, the condition of power, and the congestion on the road.

To conduct this tonnage service the Company used two classes of Mallet steam locomotives, one known as the "U.S." for road haulage over the Division, and another and heavier one known as the "A.E." type, for pushers on the grades. The general characteristics of these locomotives are given in Table 1.

TABLE 1.—Characteristics of Mallet Steam Locomotives Used on the Virginian Railway.

Type.	Wheel arrangement.	Weight of locomotive, in tons.	Weight on drivers, in tons.	Weight of tender, in tons.	Total weight in working order, in tons.	Rated tractive effort, in pounds.
U. S.	2- 8- 8-2	265.5	239	104.5	370	101 300
A. E.	2-10-10-2	342	308.5	107.1	450	147 200

The coal cars used are both hopper and gondola. Of these 7 100 are of 52 to 55 tons capacity, 500 are of 70 tons, and 2 029 of 110 tons capacity. Overloads of 10% are allowed.

The standard ratings for the trains varied in practice considerably, and were subject to a reduction of 10 to 20% in winter weather and to special reductions at other times, depending on the condition of the steam power, which cannot always be maintained 100% efficient. Furthermore, successful hauling of these extremely heavy trains over a line which is undulating in character, assumes good condition of car draft-rigging and air-brakes, and very careful handling both in starting and stopping. As would naturally be expected, these conditions are not always present and "break-in-twos" were not infrequent.

Due to all the variables mentioned and inherent in steam service, congestion and delays were frequent, especially in bad weather and at times of heavy business, hence provisions for additional facilities became imperative. It was considered that electric traction offered the best and most progressive means of securing these facilities.

Determination of Electric Traction Requirements

Knowing the physical character of the railway, the primary considerations are the power demands at the train and from the system as a whole. These are determined by the weights, speeds, and number of trains in motion at one time.

Train Weight.—Allusion has been made to the very heavy trains operated by steam locomotives, in fact, train weights had been increased nearly to

the limit of allowable stresses at the car couplings in a service conducted over broken grades. At the start of electric operation, therefore, it was believed best not to attempt a material increase in the standard weight of trains over the low-grade part of the Division, but rather to secure other possible improvements in the operation, especially to shorten the running time, to eliminate pushers east of Clarks Gap, and to handle reliably a fixed train maximum, substantially regardless of weather conditions. In effecting these operating improvements, the type of locomotive and the charactertistics of the electric system throughout should be made flexible enough to permit the running of heavier trains if conditions should so demand in the future.

The train make-up fixed upon as most suitable, all things considered, was 6 000 tons trailing weight up the hill from the assembly yard at Elmore, handled with a head engine and a pusher, this train to be filled out to 9 000 tons at Clarks Gap and handled by a head engine only for the entire 116-mile run to Roanoke. In this way the pusher formerly required on the Alleghany hill from Whitethorne to Merrimac with steam service was eliminated. This train weight and motive-power assignment effects a practical balance between locomotive adhesive requirements on the grades and the electric motor capacity needed for the long sustained runs, and does not increase car draft-rigging stresses beyond those which have been demonstrated as allowable under the physical conditions obtaining on this railway.

Speeds.—It has been stated that it was especially desired to shorten as much as possible the running time between Elmore and Roanoke, which by steam locomotives required frequently as many as 20 hours. Experience on the Norfolk and Western Railway demonstrated that it was undesirable to permit fluctuating speeds which in places might become excessive; in fact, that two standard speeds, reliably maintained, best suited a difficult track profile and alignment. On that road a speed of 14 miles per hour on the heavy grades, where complicated by excessive curvature, and 28 miles per hour on the low grade and light curvature portions of the line have proved to be suitable. In the light of this experience, and because the physical conditions on the two railways are practically the same, these standard speeds were adopted in the electrification of the Virginian Railway.

It should be noted, as will be made clear in describing the design of the locomotives, that the running speeds just mentioned are fixed. They depend only on the motor combinations and do not vary with the grades or loads. Fig. 3 shows the speed assignments adopted for an eastbound run over the Division with a loaded train. It will be seen that about 47% of the distance is covered at a speed of 28 miles per hour and the remainder, which comprises the pusher and the regenerating grades, is run at the relatively low speed of 14 miles per hour.

This set-up results in a running time of 7¾ hours over the 131 miles between Elmore and Roanoke; in practice, however, this may be extended to 9 to 11 hours, depending on the time taken for filling out the train at Clarks Gap, and for inspection at Whitethorne, which latter is the only other regular stopping place on the Division. It was predicted that under favorable condi-

tions, if not in general practice, it would be possible for an electric locomotive to make a round trip over the Division (262 miles) in 24 hours. This, in fact, has been done frequently during the winter of 1926, and the average mileage per day secured per locomotive in service, in November, was 180. This may be rightly regarded as a fine performance, when it is considered that approximately one-half the number of locomotives are used in pusher and short-run hill services.

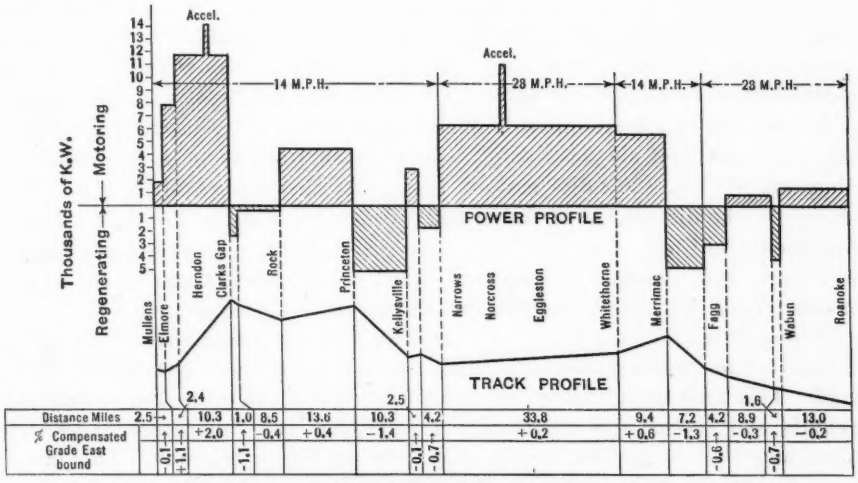


FIG. 3.—ROANOKE-MULLENS ELECTRIFICATION, VIRGINIAN RAILWAY COMPANY. POWER DATA, EASTBOUND.

Yearly Tonnage.—The railway handled 8 306 000 net tons of coal eastbound in 1925, which was an increase of 10% over that handled in 1924. In 1926 traffic took a sudden jump, due, perhaps, to unusual conditions (such as, coal mining strikes in England), and 11 500 000 tons were moved. Assuming, however, a regular and normal yearly increase in business, it was felt that the initial electric equipment should be capable of handling comfortably a total of 12 500 000 tons eastbound per year, and that the plant should be susceptible of ready enlargement to take care of a progressively increasing traffic. The ratio of average to maximum day movement is about 60 per cent. Therefore, the eastbound tonnage on a maximum day becomes 57 000, or a gross of about 80 000 tons of cars and lading. In addition, manifest and local trains and the returning light trains must be moved. A summation of all this tonnage means a total of 110 000 to be moved on a maximum day. In fact, this quantity was actually handled on certain days during 1926.

Number of Trains.—In order to meet the yearly and daily traffic demands outlined, it was estimated that for a heavy day movement fourteen 6 000-ton trains must be hauled up the heavy grade from Elmore in 24 hours. At Clarks Gap these would be combined into nine 9 000-ton trains to be hauled through to Roanoke. In addition, provision must be made for one local freight per day in each direction and a certain amount of switching duty. The empty cars are returned in nine trains of 3 000 tons each, thus balancing car and

locomotive movements eastbound and westbound. Obviously, these trains cannot be spaced evenly over the line as traffic and operating necessities require that a certain amount of "bunching" be allowed. Provision was made, therefore, to permit that two tonnage trains should operate simultaneously on the Clarks Gap Hill, spaced ½ hour apart; also, that two 9 000-ton trains should continue through to Roanoke with an interval of ¾ hour between.

Allowing for the simultaneous occurrence of westbound light movements, seven trains may be in motion and taking power at one time, of which, five may be eastbound loaded trains. At the time this installation was planned there was no westbound coal movement from points on the Division. During electrification construction, however, an Interstate Commerce Commission order resulted in a large quantity of coal (now about 300 000 tons per month) being shipped west *via* the Norfolk and Western Railway by a connection at Matoaka, W. Va. (20 miles east of Elmore). This has required a material change in the disposition of trains and locomotives and has largely increased movements on Clarks Gap Hill. The electric installation, however, has proved so flexible and adaptable to varying conditions that no changes in the power or other facilities have been found necessary on this account.

Power Demand.—In order to fix the total power requirements at the power house, it is necessary to ascertain, first, that needed to move one train of each kind. This may be calculated in the usual manner, knowing the established weights and speeds and assuming the frictional and other resistances. A curve may then be plotted for the train over the entire Division. This curve is applied to the desired train sheet to determine the total power requirements on the assumptions made as to the number and kinds of trains in motion and their spacing. Fig. 3 has been plotted to show graphically an eastbound movement of a loaded train over the line, together with the energy inputs at all points, including arbitrary starts interpolated at critical points to indicate any extraordinary energy requirements. It will be noted that a 6 000-ton train (7 285 tons weight with the two locomotives) on Clarks Gap Hill requires about 14 000 kw. at the train when starting, and the continuous input while running up this 2% grade is about 11 700 kw. The long run up the New River Valley of a 9 000-ton train (9 640 tons with one locomotive) requires about 11 000 kw. during the start and the accelerating period, and 6 500 kw. while running at 28 miles per hour.

At two places substantial amounts of power are regenerated over a considerable time interval, namely, when descending the Kelleysville and Merrimac grades, where energy is returned to the contact wire at the rate of about 5 000 kw., the train being held back in this way without the use of brakes and without an appreciable change in speed. The power regenerated during the entire eastbound run is approximately 7 800 kw-hr., which is about 20% of the total energy consumed to propel a loaded train over the Division. A westbound run with an empty train does not regenerate a large amount of power, but the total eastbound and westbound movements, it has been found in actual operation, supply by regeneration more than 8% of the total energy required to operate the road.

It has been found that generally all power regenerated is usefully employed to assist other trains which happen to be in motion, but any surplus current returned is automatically absorbed in special water rheostats at the power plant. Regeneration, furthermore, is very useful in holding smoothly and safely long trains on heavy down grades, and is of material assistance in avoiding "break-in-twos" arising from defective condition or improper application of brakes. There is also an advantage in avoiding undue wear and over-heating of brake shoes and car wheels.

Load Factor.—Obviously, under these extreme energy demands of a train, it becomes an important matter to secure as high a daily load factor as possible, that is, to space trains fairly uniformly over the line, and it would be ideal if seasonal peaks could be avoided. If these ends could be effected, they would result in the greatest possible economy of power and would reduce the amount of electric equipment of the entire installation to the minimum.

It is obvious, however, that uniformity in demand cannot be secured even approximately in practical railroading. Business does not come to the railroad in such a way as to permit it. It would be expensive to provide, for this purpose, yards large enough to hold trains for uniform dispatching. The best that can be done is to take business as it comes, with judgment exercised to prevent undue "bunching" of trains in the various sections of the line to an extent that will overload the power plant and sub-stations. Train sheets for normal and, therefore, extreme conditions were laid down to satisfy practical operation as indicated by experience, always having due regard to the factors producing reasonable plant cost and economy of operation.

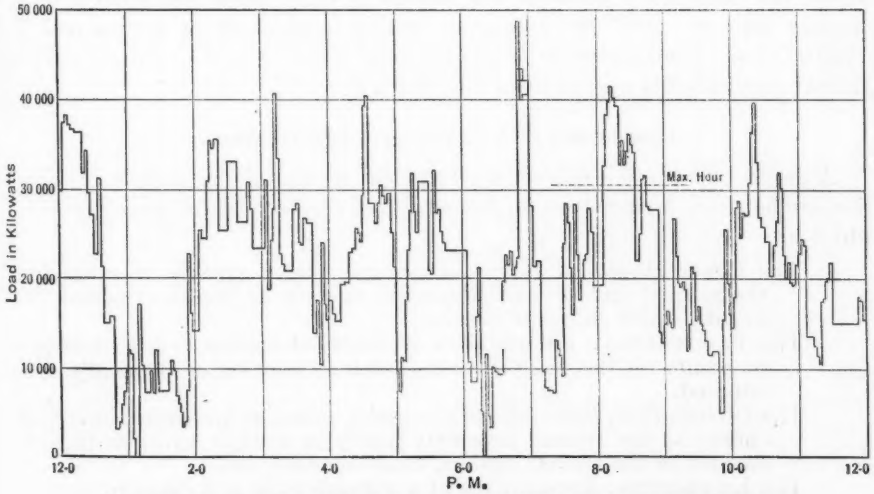


FIG. 4.—TYPICAL HEAVY DAY LOAD, IN KILOWATTS, AT POWER HOUSE.

Daily Load Curve.—A 12-hour curve of loads at the power house, developed from the factors mentioned, for a congested day is given in Fig. 4. It indicates very heavy fluctuations in demand, not only over the hour, but during short intervals. Thus, the maximum 5-min. peaks are of the order of 40 000

to 45 000 kw. and the hourly peaks are about 31 000 kw. Furthermore, the entire load drops at times almost to zero. Under the full electrical operation the peak loads have been substantially as calculated, showing generally a 50% daily load factor, referred to the maximum hour, and at times the load factor has been as high as 65 per cent. As traffic becomes heavier, due to the operation of more trains, the daily average load factor will become greater, but it is estimated that the peak loads will not materially increase.

The loads thus developed at the power house for a normal and for a maximum congested day are shown more clearly in Table 2.

TABLE 2.

Loads.	Normal day, in kilowatts.	Maximum congested day, in kilowatts.
Average load for 24 hours	14 500	20 500
2-hour peak	22 300	29 200
1-hour peak	31 300	31 500
5-min. peak	38 000	43 500

The average daily kilowatt-hour station output was estimated to be about 310 000; the actual output, when handling the volume of business estimated, has averaged 325 000 kw-hr. The yearly station output, of course, depends on the actual tonnage moved. Based on handling a total net coal tonnage of 12 500 000, it was anticipated that the station output would be about 112 000 000 kw-hr. per year. In a period of six months of full electric operation a total trailing load of 10 037 000 tons was hauled, equivalent to approximately 5 500 000 tons of coal, with an average haul of about 96 miles. In this period the net power-station output was 51 300 000 kw-hr.

Description of the Electric Installation

A general description only of the technical aspects of the electric installation can be given, laying stress on the conditions determining its main features, which are:

The Power House: A source of primary power capable of satisfying the normal and seasonal demands, as well as the short-period demands, which fluctuate greatly.

The Transmission: Transmission of electrical energy at high voltages to points on the railway from which it may be economically distributed.

The Distribution: Sub-stations at suitable points to transform electrical energy to the voltage required; trolley or contact wires to furnish current to the trains; feeders, track bonding, etc.

The Locomotives: Locomotives of a suitable type and capacity.

All these features must be designed so as to conduct service successfully irrespective of weather or other variable conditions and in a manner consistent with low installation, operating, and maintenance costs. Furthermore, the system must be such that additions can be readily made to it as traffic grows, and that improvements in apparatus, due to advancement in the

art, may be incorporated without radical and expensive changes in the installation as a whole.

Practical railway operation requires that there shall always be enough equipment provided and power available to permit both normal and congested movements, regardless of the peak loads produced at any point, that is, that traffic movement shall not be subordinated to equipment limitations. Therefore, in determining the quantities of apparatus, this general principle was followed.

The successful development of heavy electric traction for railways is due primarily to advances in the art, which permitted great amounts of power to be transmitted over long distances. As electrical energy is the product of pressure and volume of current, it follows that the higher the pressure the smaller the current required to transmit a given amount of energy, hence the use of very high voltages for power distribution. In early electric traction installations, the distances involved were comparatively short; therefore, moderately high transmission voltages were sufficient for economical results. The distribution voltage in the feeder and contact systems could also be low without unduly increasing the cost of the conductors.

As the distance to be covered increased, transmission voltages were raised for economic reasons, but a more important and difficult problem had to be solved in devising a suitable current-collecting system for the trains. The steel rail formerly used as a contact member necessarily was replaced by an over-head contact wire in which high voltages could be used. Furthermore, new types of motors were developed to utilize high voltages. Logically, the ultimate aim is to find a general system that will permit unrestricted voltages in the collector wire (trolley) to suit economic requirements for any and all distances and traffic. To effect this result, in the present state of the electrical art, it appears necessary to fix the trolley voltage at a point high enough to satisfy the most extreme conditions and in some manner to lower the voltage on the train to suit practical motor and control development.

The fundamental distinction between electric systems now available, therefore, may be expressed in terms of the kind of current used in the trolley wire, that is, "alternating", or "direct". The alternating-current system is the nearest approach so far to a general one; it has been used very successfully for the past fourteen years on the Norfolk and Western Railway, under physical and traffic conditions similar to those of the Virginian Railway and on many other railways in America and abroad. Its adoption on the Virginian Railway was, therefore, a logical procedure, both on account of its demonstrated success, and because the railway lies closely adjacent to the Norfolk and Western and there is an interchange of power and traffic between the two.

The system referred to is generally termed the "single-phase alternating-current", but in this instance three-phase motors are used on the locomotives to obtain certain advantages of this type for the specified haulage conditions, as will be explained later. Single-phase alternating-current, or even direct-current, motors, however, can be used on the locomotives without changes in other parts of the traction system, should it be desired to do so in the future in order to meet changed business conditions.

The main elements and special features of the Virginian electrification will be described, beginning with the source of the power.

The Power Plant

Source of Power.—It is sometimes assumed that an important consideration in determining upon a railway electrification is that advantage can be taken of cheap hydro-electric power. Unfortunately, in the Eastern United States, available water powers are few, and such as exist are not generally near the consumer, and are often not dependable in capacity, so they must be assisted by some form of fuel-burning power plant. Where, therefore, as in the case of the Virginian Railway, high-grade coal is close at hand, a steam plant is suitable and economical, all things considered. It is a reliable means of supply at all times and can be so placed as to meet the needs of the Railway Company.

As an alternative to the construction of a railway power plant for the traction supply, examination was made of the possibility of purchasing power from an existing commercial plant. If power could be secured in this way at a cost not materially in excess of the generating cost of the Company's own plant plus fixed charges, interest, depreciation, and taxes, it would be advantageous, of course, to purchase power and thus save an investment of $4 000 000 in a new plant.

Four different possibilities were examined, but only one commercial power company was prepared to submit a definite proposition and this was so disappointing as to cost, other terms, and conditions, that it was impossible to give it further consideration. As a matter of fact, the load conditions already outlined are so exceptional that it is doubtful whether any existing local company engaged in supplying electric energy to the general public could afford to take on a load of this character, except by devoting special generators and transmission circuits to the railway supply, and thus being under a special handicap in large fixed charges.

Doubtless, there is a field for commercial power companies, especially large networks, in the supply of power for heavy railway traction, but the terms and conditions must meet the ordinary and extraordinary loads of the railway and in order to compete such companies must be satisfied with a low margin of profit, as low power cost is depended upon to furnish the largest direct return on the investment in a railway electrification.

Plant Location.—It will be noted from Fig. 1 that the railway is located in the valley of the New River along which it follows for many miles. This stream varies greatly in volume of flow, but contains at all times an ample supply of water for boiler-feed and condensing purposes. The U. S. Government records for the past 40 years show a water-stage variation of 30 ft. in this river, with maximum floods in 1870 and 1916.

A careful survey was made to determine a suitable power-house site, in view of these high and low-water conditions, at a place where the construction would not restrict the flood prism. Furthermore, it was important to select a site near an established town where employees could be housed. All

conditions were best met at the Town of Narrows, Va., 74 miles west of Roanoke. There, the river valley broadens, and a bend occurs, forming a wide and level plot alongside of which the railway runs at a suitable elevation for connections and sidings to deliver coal and materials to the plant.

General Character.—The plant was designed to insure continuity of service under widely fluctuating loads. Good operating economy was aimed at, but refinements were not included to the extent of unduly complicating apparatus and increasing maintenance difficulties. This seemed to be a conservative procedure, as the cost of fuel is low compared with that found at commercial plants more remote from the coal fields, and as the plant is situated at a considerable distance from skilled labor markets.

There was one consideration which gave the designers concern, namely, the unprecedently great spread between maximum and minimum short-period demands on the steam-generating features of the plant. At certain times, the load at the power house drops off practically to zero (see Fig. 4). At others, there may be only one train on the line taking power and, at still others, there may be four trains, or more. As the instantaneous increments of load may be of the order of 14 000 kw., it is obvious that unusually severe requirements are imposed on the boilers in the production of steam in suitable volume at substantially constant pressure. This same problem had been faced to a lesser degree in the power plant of the Norfolk and Western Railway where the undesirable limitations of the ordinary form of boiler-stoking had been demonstrated.

Therefore, the much more severe load conditions on the Virginian plant appeared to make a change in firing practice desirable and, perhaps, essential. Liquid or gaseous fuel would be ideal, but powdered coal fuel, which has the burning quality of a gas, appeared to offer the most practical solution of the difficulties. The success of powdered fuel had been demonstrated in a number of plants under a variety of conditions elsewhere, thus its use here appeared not wholly experimental. Incidentally, it promised the highest obtainable fuel economy, but its ease of regulation was the main factor determining its use for the Virginian installation.

It is gratifying to be able to state that in the practical operation of this plant for more than 1½ years, the wisdom of using powdered coal has been demonstrated as producing entirely manageable boiler firing under the normal and extreme conditions imposed. The station coal rate has averaged less than 1¾ lb. per kw-hr. of net output, or about 14½% efficiency, which, considering the character of the load, is a satisfactory showing. The installed capacity of the power plant was set at 48 000 kw., 1-hour rating, divided into four generating units and five boilers, designed to provide a spare for each.

Building and Foundations.—The local rock is limestone of variable quality, so bedded as to be subject to erosion by underground waters. As the concentrated column loadings were necessarily heavy, special care was taken to explore the site thoroughly by borings to determine sub-surface conditions. Considerable variation in the quality of the rock was found and, unfortunately, the disintegrated portions occurred at certain heavily loaded points.

No cavities were found below sub-grade, but it was thought advisable to establish the low unit pressure of 3 tons per sq. ft. for the heavily loaded stack footings.

In order to establish a reasonable height for the main floor of the building to permit of ready access to the plant, and at the same time to provide economical conditions in the supply of condensing water, the elevation of the basement floor was fixed 30 ft. below high-water mark. This necessitated very substantial walls and floors under the turbine room to withstand the hydrostatic head during flood stages. As the displacement at the basement produced an uplift under these conditions in excess of the total weight of the construction above, it was necessary to insure against any possible movement at such times. This was done by providing a reinforced basement floor slab having concrete ribs between the piers of the turbine foundations, and by anchoring these piers with rods 1½ in. in diameter grouted into the rock. Of these anchors, 134 were used. To insure further against any uplift movement, vents were provided in the concrete slab to relieve any pressure which might build up a hydrostatic head.

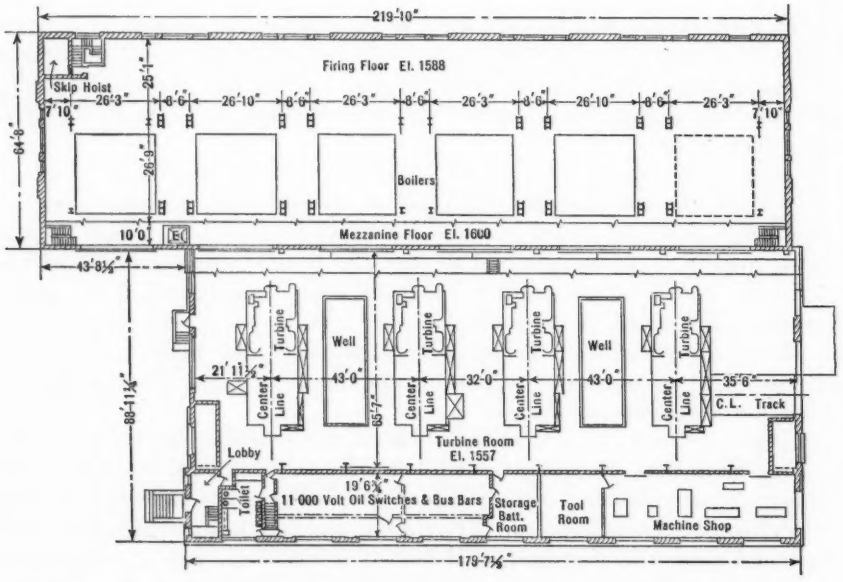


FIG. 5.—FLOOR PLAN, TURBINE ROOM AND FIRING FLOOR, NARROWS POWER PLANT, VIRGINIAN RAILWAY.

The general character and arrangement of the building is shown in the plan and cross-section, Figs. 5 and 6 and in the photograph, Fig. 8. The dimensions and other data of general interest are given in Table 3. The building walls are of brick and are self-supporting; the basement walls are of reinforced concrete. Interior steel framework carries the floors, crane, stacks, and the boiler-room apparatus.

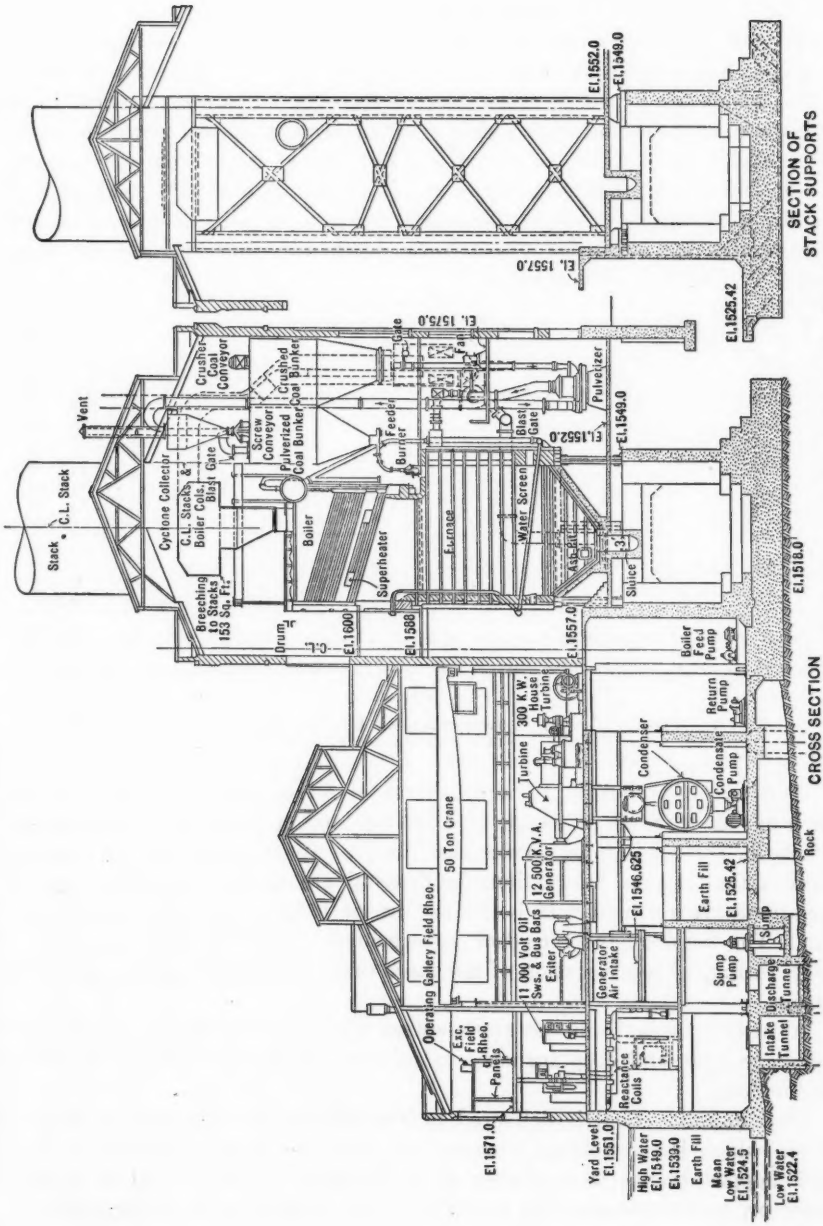


FIG. 6.—PLAN AND CROSS-SECTIONS, NARROWS POWER PLANT, VIRGINIAN RAILWAY.

Stacks.—The two stacks (Fig. 8) are very high, and are constructed of reinforced concrete, supported on, and directly bonded into, concrete mats at the roof level of the boiler-room. Each stack base is supported by four steel columns, which also serve as boiler supports, and are wind-braced to adjacent boiler columns by flexible members. These permit sway in the supports, which are kept free from the roof framing. These main stack foundations rest on concrete piers which, in turn, rest on spread concrete footings. The stacks are 25 ft. 8 in. in outside diameter at the bottom and 16½ ft. in inside diameter at the top, and rise 285 ft. above the roof mats, making a total height of 376 ft. above the bottom of the ash-pits. The stacks were designed to handle high boiler ratings with natural draft only, thus avoiding the complication of forced draft apparatus.

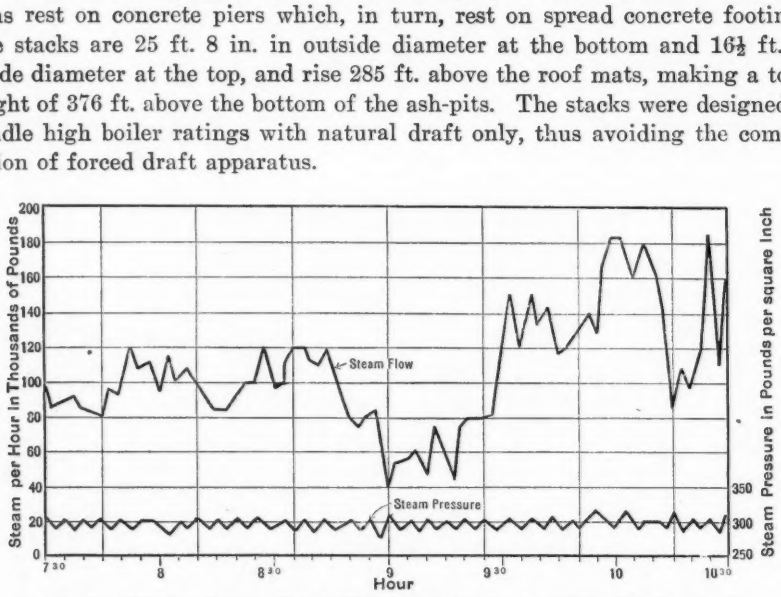


FIG. 7.—TYPICAL STEAM FLOW FROM ONE BOILER, NARROWS POWER PLANT, VIRGINIAN RAILWAY.

Boilers.—Five boilers of the horizontal water-tube type, are provided. Each has an A. S. M. E. rating of 1 520 h.p. and operates at a steam pressure of 325 lb. per sq. in. In service these boilers are at times run at very high ratings, that is, the fluctuations in demand are violent (see Fig. 7). The variations are from normal boiler rating, or less, up to 400% rating in 2½ min. The boilers have interdeck superheaters to furnish 150° superheat at 160% rating, with a variation of not more than 15° between normal and 440% rating.

As will be seen from the cross-section (Fig. 6), the boilers are mounted over very large and deep furnaces to suit the requirements of powdered fuel combustion.

Coal Supply.—Coal is delivered by the railway in cars from a main-line siding, which has a holding capacity of thirty-five cars. Delivery is into a track hopper, thence to a crusher by a belt conveyor fitted with a magnetic separator, to remove metallic particles. The crushed coal is then lifted by a skip-hoist into bins in the boiler-house, having a total storage capacity of 900 tons. There is also a yard storage space for 2 000 tons, to and from which coal is readily piled and reclaimed by mechanical means.

Pulverized Fuel.—The equipment used to provide powdered fuel takes the crushed coal from bunkers in the power-house building by gravity through spouts into pulverizing mills on the ground floor. The mills grind the coal to a size passing a 150 mesh; delivery is through a pipe to the top of the building by air pressure (see Figs. 6 and 9) and into a cyclone separator. From the latter it drops into a horizontal conveyor in which a screw operates to deliver the pulverized coal into bunkers above the firing floor. One bunker, provided for each boiler unit, has a capacity of about 130 tons, or about one day's supply. From these bunkers the pulverized coal is delivered by screw feeders to ten jet burners for each boiler, so mounted as to inject the fuel into the furnace in divided and nearly vertical streams.

Air under pressure is admitted with the coal into the burner nozzles and acts as a vehicle to carry the finely divided fuel into the furnace at the proper velocity. Fig. 10 is a view of the mechanism for controlling the speed of the burners and the draft. A certain amount of air is drawn in around the nozzles, but the main supply for combustion is introduced through inlets in the walls of the furnace so arranged as to furnish each particle of burning coal with sufficient air for its complete combustion. For the purpose of avoiding excessive temperatures in the brickwork, the furnace side-walls are made hollow, forming a space through which the greater part of the air for combustion is circulated.

The jet flame travels downward toward a horizontal water screen in the lower zone of the furnace where its injection velocity is lost. The flameway then doubles back on itself and is drawn upward at the rear of the furnace by the stack draft over and across the boiler tubes. The jets travel in the furnace about 50 ft. before reaching the boiler-tube surface, thus allowing sufficient time for complete combustion in the furnace proper. To secure this long flameway, of course, the furnace must be very large; it is of about 14 000 cu. ft. contents, or 9.2 cu. ft. per rated boiler h.p.

A certain amount of tubular heating surface is provided in the water screens near the rear wall and across the bottom of the furnace; this serves to protect the lining from the excessive temperatures produced at points where the fuel jet impinges on the walls. The tubes at the bottom also act to prevent the formation of slag, and cause the ash to take the form of small particles which are projected into a pit, from which they are carried away by a stream of water flowing in a sluice-way. Because of doubt as to the most suitable location and amount of furnace side tubing for this form of combustion, a limited heating surface only was supplied in this way. It appears probable that eventually the furnace for powdered coal combustion may contain the major portion of the total boiler-heating surface; that is, this form of firing may lead to a radical change in water-tube boiler design.

The control of combustion in the furnace under varying requirements is effected by regulating the amount of fuel to the burner and also the air supply. Furthermore, individual burners can be "cut in or out" as desired. The combustion is controlled partly automatically and partly by hand. Each boiler is equipped with a control box which contains a system of gearing and

switches to vary the speed of the fuel feed and the movement of the dampers to admit air to the furnace. This apparatus (Fig. 10) is, in turn, controlled from a master switch on the central panel board on which are mounted temperature, pressure, and draft gauges.

The variation of steam pressure in the boiler header actuates a master switch, which regulates movements in the individual control boxes at the boilers. Push-button switches are also provided on the panel to permit of "cutting in and out" individual jets and the adjustment of air and fuel by hand, as required. Thus, the amount of fuel in the furnace at any time can be made to follow closely the plant load conditions and, therefore, the variation in steam pressure can be more uniformly maintained than is possible where there is a large amount of partly consumed solid fuel in the furnace.

As illustrating what has been accomplished in actual practice with this form of firing, Fig. 7 gives a record of the steam-flow meter chart on one of the boilers. The upper line shows that steam requirements varied from about normal to 400% of rating. The lower line gives the record of pressure in the saturated steam header, which fluctuates about 25 lb. under extreme conditions. This is a very satisfactory showing. In practice, it is seldom that the boiler safety valves open on account of excessive rise in pressure.

Steam Turbines.—Four turbo-generators are provided. These have cast steel bed-plates mounted on box girders filled with concrete and supported on concrete piers, thoroughly cross-braced. This heavy monolithic structure is designed to eliminate vibration. Each turbine is furnished with its own condenser (mounted underneath), of the "surface" type, having 20 000 sq. ft. of tube cooling surface. The water supply, amounting to about 20 000 gal. per min. for each condenser, is obtained through an intake tunnel at the upstream side of the power house and discharged through an outflow tunnel down stream.

Each turbine is direct-connected to a generator. This machine is wound for 3-phase 25-cycle current at 11 000 volts. It has a continuous capacity of 10 000 kw. at 80% power factor, single-phase rating. Its single-phase rating for 1 hour is 12 500 kw. A view of the turbine room is shown, Fig. 11.

Transformers.—From the generators the current passes through disconnecting switches to the step-up transformers, which are single-phase and raise the potential to 88 000 volts. Thence, it passes through oil-circuit breakers to the high-tension transmission lines, which deliver current to the sub-stations along the railroad. The step-up transformers, lightning protection, and other sub-station apparatus are placed outside the power-plant building (Fig. 8). The transformers are four in number and are oil-immersed and water-cooled. Each is rated at 10 000 kv-a. continuously, or 15 000 kv-a. for 1 hour, and 20 000 kv-a. for 5 min. following a continuous full-load run. The system power factor is about 85 per cent.

Regenerating Rheostats.—As has been mentioned, on heavy down grades at several places along the railway, a descending train automatically returns electrical energy to the distribution system. Should there be other trains in motion elsewhere and drawing current, the energy thus returned assists in

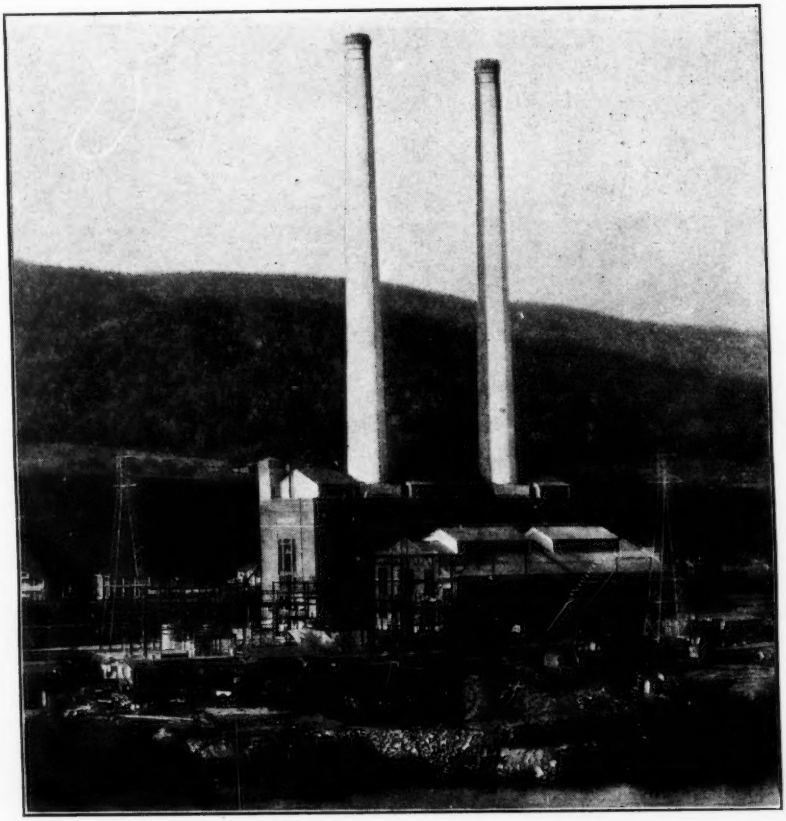


FIG. 8.—GENERAL VIEW OF NARROWS POWER PLANT, INCLUDING SUB-STATION.

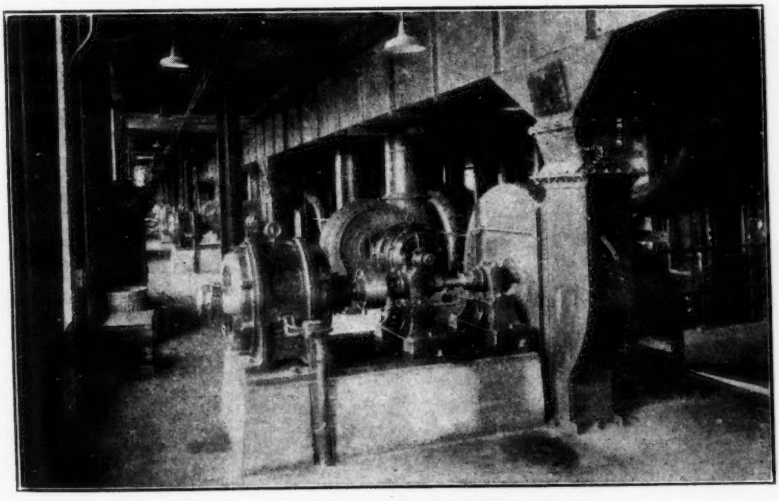
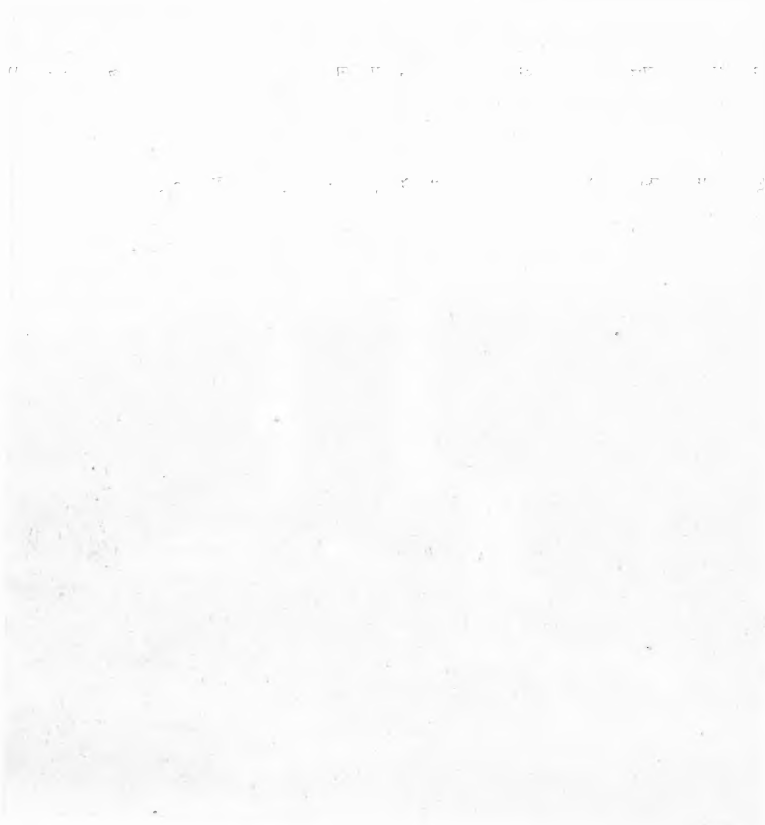


FIG. 9.—NARROWS POWER PLANT: VIEW OF BLOWER FOR SUPPLYING AIR TO PULVERIZED FUEL BURNERS.



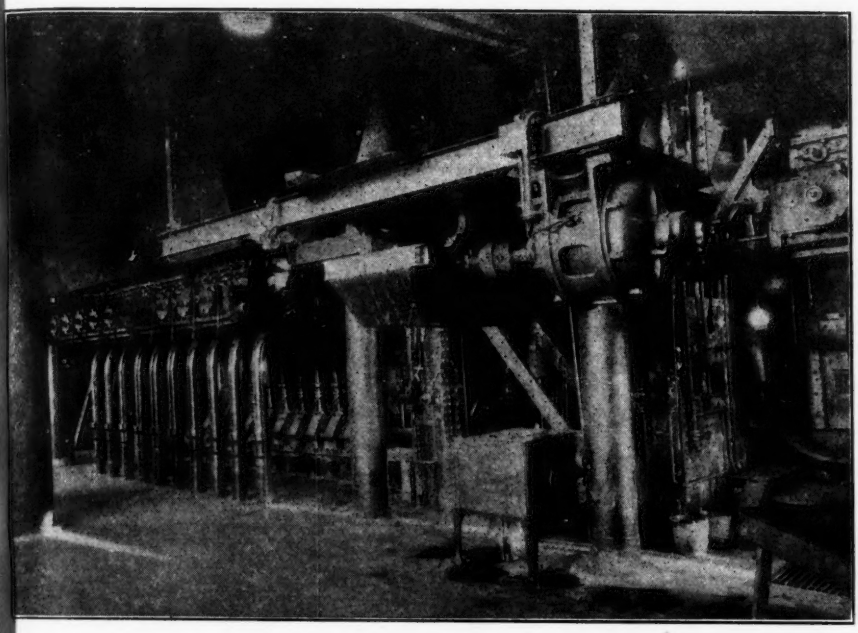


FIG. 10.—MECHANISM FOR CONTROLLING SPEED OF BURNERS AND DRAFT, NARROWS POWER PLANT.

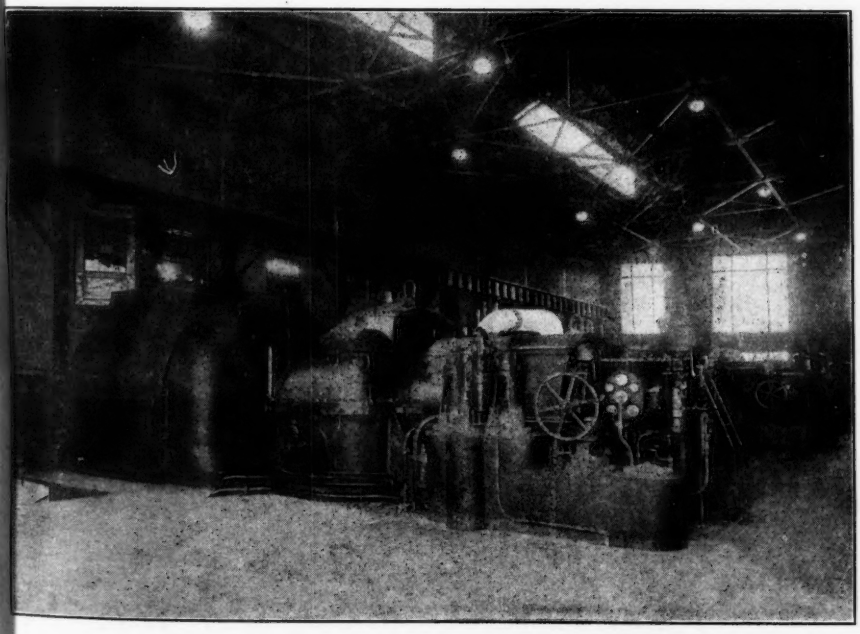


FIG. 11.—VIEW OF TURBINE ROOM, NARROWS POWER PLANT.

TABLE 3.—Narrows Power Plant. General Data.

(Installed generating capacity (continuous rating), 40 000 kw.; output per year to move 12 500 000 tons of coal, 120 000 000 kw-hr.)

Building dimensions	Boiler room	217 ft. long, 62 ft. wide.
	Turbine room	176 ft. long, 65 ft. wide.
Areas	Boiler room	13 500 sq. ft.
	Square foot per boiler horse power	1.48 sq. ft.
	Square foot per kilowatt generators installed	0.34 sq. ft.
	Turbine room	11 500 sq. ft.
	Square foot per kilowatt generators installed	0.29 sq. ft.
Elevations above sea level	Basement floor	1 525.0 ft.
	Extreme high water	1 549.0 ft.
Stacks	Height above ash-pits	376 ft.
	Height above concrete mat at roof	285 ft.
	Top inside diameter	16 ft.
Fuel used (coal)	Quality	West Virginia low volatile 13 500 B. t. u.
	Average daily consumption	250 tons (run of mine).
	Capable of handling machinery	85 tons per hour.
	Outside storage on track	30 cars (3 000 tons).
	Outside storage yard	2 000 tons.
	Inside storage, crushed	900 tons.
	Inside storage, pulverized	650 tons.
Boilers	Number installed	Five; space for one additional.
	Rating, each	1 520 b. h. p., A. S. M. E.
	Heating surface, each	15 210 sq. ft.
	Furnace dimensions	26 ft. 10 in. long, 16 ft. 3 in. wide, and 24 ft. 6 in. high.
	Superheater surface, each boiler	1 770 sq. ft.
	Pulverized fuel burners, per boiler	10
Turbines	Number installed	Four; space for one additional.
	Type	Parallel flow, impulse reaction.
	Speed	1 500 rev. per min.
	Operation	275 lb. steam pressure, 150° superheat, vacuum, 29 in.
	Maximum economy	12.48 lb. of steam per kw-hr.
Generators	Continuous rating, each	10 000-kw., 25-cycle, 3-phase, 11 000-volt.
	Exciters, direct-connected	125-kw., 250-volt, direct-current.
Condensers	Number installed	Four.
	Type	Surface; two-pass.
	Tube surface, each	20 000 sq. ft.
	Condensation water required, each	20 000 gal. per min.
Circulating pumps	Number installed	Four.
	Type	Centrifugal, with direct-connected variable speed, alternating-current motors.
	Capacity, each	20 000 gal. per min.
Condensate pumps	Number installed	Four.
	Type	Centrifugal, with direct-connected alternating-current motor.
	Capacity, each	300 000 lb. per hour, against 135-ft. head.
Boiler feed pumps	Number installed	Three.
	Type	6 in., four-stage, steam, turbine-driven.
	Capacity	650 gal. per min.
Feed-water heaters (open)	Number installed	Two.
	Type	Metering.
	Capacity, each	300 000 lb. per hour.
Feed-water heaters (bleeder)	Number installed	Four.
	Type	Closed.
	Capacity, each	200 000 lb. per hour.
House service water supply	Filters	Three, vertical pressure type.
	Capacity of each filter	350 gal. per min.
	Filter wash tank	One.
	Capacity of tank	7 500 gal.
Auxiliary power turbo-generator	Turbine	One unit, operating 6 000 rev. per min.
	Generator, gear-connected	300-kw., 440-volt, 3-phase, 750 rev. per min.
	Exciter, direct-connected to generator	100-kw., 250-volt, 750 rev. per min., direct-current.

TABLE 3.—(*Continued.*)

Air compressor, vacuum cleaner system	Installed capacity	One, 150 cu. ft., motor-driven, 100-lb. pressure.
	Type	One, vacuum producer, inlets on several boiler-room floors.
Crane	Installed capacity	50 tons with 10-ton auxiliary hoist.
	Type	One electric traveling, in turbine room.

propelling them. Should there, however, be current returned in excess of the amount required by other trains, this excess must be absorbed in some manner to prevent overspeeding the generators at the power house. For this purpose, water rheostats are provided. These are single-pole electrodes placed in the water at the mouth of the discharge tunnel. They cut in automatically and quietly absorb the surplus energy. It is found in practice that these rheostats seldom come into action, because over a long railway line generally a train is in motion to make use of all regenerated energy.

As the foregoing description of the power plant is very brief, additional statistical information is appended in Table 3.

High-Tension Transmission

From the step-up transformers at the power house a main transmission line distributes current to the sub-stations along the railway. Such transmission circuits are generally carried either on the poles or bridges supporting the catenary wire system, or a separate pole line may be located on the right of way. The Virginian Railway is a single-track line, except for 22 miles at the west end. The right of way is narrow, with a great amount of curvature, and is complicated with many tunnels and high viaducts. Moreover, future track changes are probable.

It was decided, therefore, to carry the transmission across country on a separate right of way, planned so as to intersect the railway only at the sub-stations, which are spaced about 20 miles apart. By reason of the rather circuitous alignment of the railway through the mountain passes, it was found possible in this way to secure a direct line transmission 34 miles shorter than the route along the railway right of way. At the westerly end, on the 22-mile double-track section, however, the transmission is carried on the catenary poles. At Matoaka, Va., a tie transmission line 9 miles long was run across country to connect with the Norfolk and Western Railway electric system at Bluestone, Va., thus tying together the power systems of the two companies for use in emergencies, or for the interchange of power.

Towers.—The line traverses a very broken country with many ridges and deep valleys which are heavily wooded. Economy of construction, therefore, required long wire spans. These are, normally, 900 ft. long, but a number are of considerably greater length, the longest being 2 440 ft. Locations for the towers were selected, where possible, upon local ridge crests, thus allowing the desired wire sag with a moderate tower height without encroaching on safe clearances at mid-span.

Square towers of the lattice type were used. These towers are of galvanized steel, and the framing and panel lengths are made suitable for economical transportation and setting up. They were designed to resist the unbalanced stresses caused by two broken wires and the tensions were limited by allowing the wires to slip in the supporting clamps. To the stresses produced under these assumptions was added that caused by a wind pressure of 8 lb. per sq. ft. acting on the projected area of ice-covered conductors plus 12 lb. per sq. ft. on 1½ times the projected area of one tower face. Ice loading of ½ in. radially was assumed as occurring simultaneously with maximum loading and wind pressures. Fig. 12 shows the design of the 88 000-volt. suspension towers.

The tower footings consist of stubs and grillages embedded in the ground to provide adequate resistance against uplift, and designed for a low unit value for bearing resistance against settlement, with due allowances for secondary strains, which might develop due to deflections in newly tamped earth back-fill.

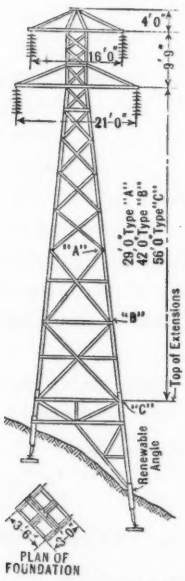


FIG. 12.—PLAN OF 88 000-VOLT SUSPENSION TOWER.

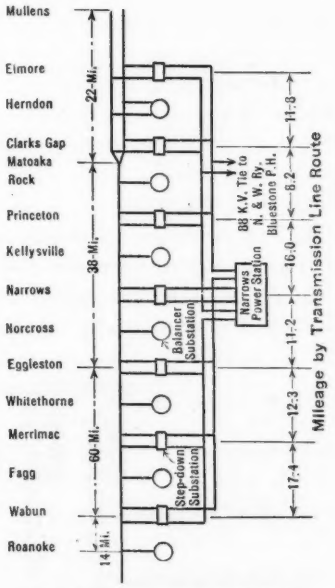


FIG. 13.—DIAGRAM OF POWER DISTRIBUTION SYSTEM.

Circuits.—The towers carry two single-phase 88 000-volt circuits, with the wires, arranged vertically, two on each side. The conductors are of hard-drawn stranded copper, having the following equivalent section:

For the 41 miles of transmission east of Narrows Power-House .....3/0 copper
For the 16 miles westward to Princeton...250 000 cir. mils.
For the 26.5 miles from Princeton to Elmore.....4/0 copper

On the upper cross-arms, the wires are spaced 16 ft. apart horizontally and, to prevent the possibility of the lower wires whipping and striking the upper

ones, those on the lower cross-arm are normally spaced 21 ft. apart horizontally. This separation is particularly necessary at the summits of the mountains where during the winter there is danger of "frozen fog", which is a crystalline mass, often of large diameter and of about the consistency of snow, forming on the conductors. The part of this mass struck by sunlight melts quickly and falls off, whereas the part in the shade remains, thus producing an unbalanced load and causing the cable to whip upward. This would require great vertical separation in order to avoid short circuits if the two cables on one side of the tower were in the same vertical plane.

The conductors are suspended from a string of seven porcelain disk insulators, with clamps so designed that slippage will occur at about 1 000-lb. pull in case of a broken conductor. Anchor towers are provided at crossings over railroads and also where necessary to take dead-end stresses. Miscellaneous data for the transmission line are as follows:

Total length:		
On towers	68	miles
On catenary structures	15	"
Total	83	miles
Number of towers	377	
Weight of steel in towers	1 816	tons
Transmission cable	332	miles
Weight of copper	530	tons

Patrol Telephone Line.—A telephone circuit for patrol purposes was added to the existing wood-pole telephone and telegraph lines along the railway right of way. In order to secure patrol telephone stations at intervals of about three miles on the cross-country transmission, laterals were run from the railway.

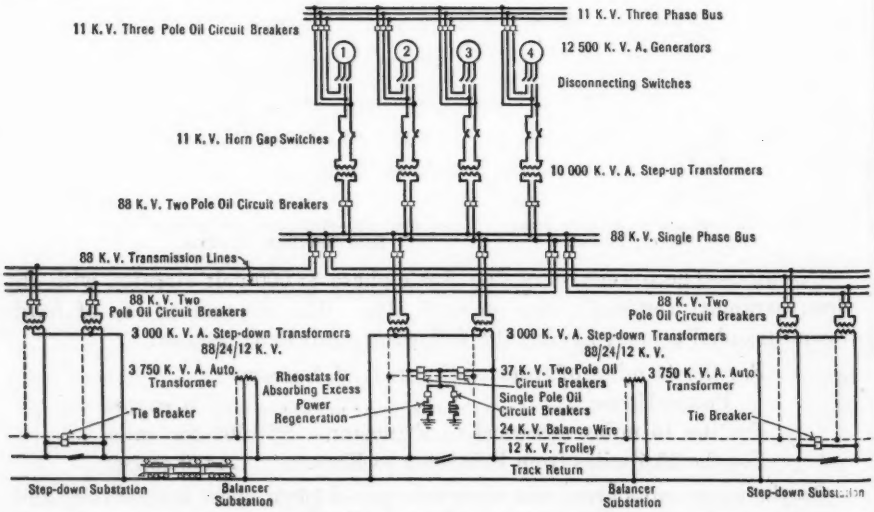

11 K. V. Three Pole Oil Circuit Breakers
11 K. V. Three Phase Bus
12 500 K. V. A. Generators
Disconnecting Switches
11 K. V. Horn Gap Switches
10 000 K. V. A. Step-up Transformers
88 K. V. Two Pole Oil Circuit Breakers
88 K. V. Single Phase Bus
88 K. V. Transmission Lines
88 K. V. Two Pole Oil Circuit Breakers
3 000 K. V. A. Step-down Transformers 88/24/12 K. V.
3 750 K. V. A. Auto-Transformer
Rheostats for Absorbing Excess Power Regeneration
Tie Breaker
37 K. V. Two Pole Oil Circuit Breakers
Single Pole Oil Circuit Breakers
24 K. V. Balance Wire
12 K. V. Trolley
Track Return
Step-down Substation
Balancer Substation

FIG. 14.—DIAGRAM OF WIRING AND CONNECTING APPARATUS FROM POWER HOUSE TO TROLLEY WIRE AND RETURN, VIRGINIAN RAILWAY ELECTRIFICATION.

Distribution

The distributing wiring and apparatus comprehends the various features of the traction system, beginning at the points where the transmission is tapped into the sub-stations. It includes the switching facilities, the transformers, the catenary-supported contact, the balancing wires over the tracks, and also the track-bonding and return conductors.

The general scheme of distribution is shown in Fig. 13 giving the relative location of the power plant and the transformer sub-stations, which are of two classes, "main step-down" and intermediate auto-transformer (or balancer) stations. Fig. 14 shows diagrammatically the complete electrical connections from the power plant to the train.

Attention is especially called to the simplicity of the switching arrangement. At the power house the step-up transformers are semi-permanently connected to the generators and all switching is done at 88 000 volts. At the sub-stations the transformers are connected directly to the trolley and balancing wire, and the principal switching is likewise done at 88 000 volts. One 11 000-volt breaker in each sub-station is used to tie trolleys and balancing wires together in emergencies, normal operation being to keep them separate in each direction from the sub-station.

In this distribution system the "balancing" wire and "balancing transformers" are unusual features. The arrangement was used for the first time on this work to effect a maximum separation distance between the step-down sub-stations and, at the same time, to reduce induction in near-by communication circuits. It was regarded as the safest and most efficient means of accomplishing the latter purpose in an installation having very large concentrated power demand normally and unusual short-circuit conditions.

Sub-Stations.—There are seven main sub-stations spaced on an average about 20 miles apart (see Fig. 13). These function to step-down the 88-kv. current to the trolley working voltage (11 000 volts) and to provide switching and sectionalizing for the catenary lines. The stations contain no rotating or other apparatus requiring regular attendants for operation. The transformers are of the self-cooled type and the switches, lightning arresters, etc., are of types, which need no housing for protection from the weather. Therefore, the stations can be made "out-of-door". The general arrangement may be seen in the view, Fig. 15. All these sub-stations are situated near a telegraph office or signal tower, where an employee is on regular duty and can be called on in emergency to open or close the oil-circuit breakers, under telephone instruction of the Power Director. Statistics regarding these main transformer sub-stations, also the balancing transformer sub-stations, are given in Table 4.

The seven balancing transformer stations are located approximately midway between the main sub-stations, and contain auto-transformers, non-automatic disconnect switches, and lightning arresters only. They have no connection with the 88-kv. transmission, in fact, they cannot be considered sub-stations in the ordinary sense of the word.

TABLE 4.—Data Relative to Transformer Sub-Stations.

	Distance from West End of Division.		
	Miles.	Number of transformers.	Total transformer capacity, in kilovolt-amperes.
Main Transformer Sub-Stations:			
Elmore, W. Va.	2.6	3	9 000
Clarks Gap, W. Va.	16.1	3	9 000
Princeton, W. Va.	36.0	2	6 000
Narrows (step-down), Va.	69.6	2	7 500
Eggleston, Va.	78.5	2	6 000
Merrimac, Va.	98.3	2	6 000
Wabun, Va.	120.0	3	9 000
Balancing Transformer Sub-Stations:			
Herndon, W. Va.	9.0	3	11 250
Rock, W. Va.	25.3	1	3 750
Kellysville, W. Va.	49.0	1	3 750
Norcross, Va.	67.6	2	7 500
Whitethorne, Va.	89.0	2	7 500
Fagg, Va.	106.2	1	3 750
Roanoke, Va.	133.0	2	7 500

An important departure from usual practice is in the type of transformer used. It is equipped with a third coil on the low-tension side. This coil furnishes current of double the trolley voltage to a "balancing" wire, which is carried on the catenary poles, and connects all main sub-stations and also the auto-transformers located at seven places along the line, midway between the main and at the easterly end of the contact line. While, in one sense, these auto-transformers are auxiliary sub-stations, their function is, in fact, quite different, as will appear subsequently. Their general arrangement and appearance is shown in Fig. 16.

Balancing System.—The essential elements of this novel form of distribution are shown in Figs. 13 and 14. They are: The three-coil transformers in the main sub-stations; the auto-transformer sub-stations; and the connecting "balancing" wire.

The three-coil step-down transformer has one of its secondary windings connected between the trolley wire and the track, and furnishes the propulsion current at 12 000 volts (11 000 volts, nominal); the other secondary is connected between the balance wire and the track and carries balancing current at 24 000 volts. The contact wire, the track, and the balancing wire thus form a three-wire system having a connecting feeder of double the trolley voltage. The reasons for the higher feeder voltage are: First, higher auxiliary transmission voltage of about 36 000, resulting in economy of copper and permitting greater through feed between sub-stations; second, the contemplated ultimate use of 24 000 volts at the trolley so that the feeder voltage, which is of opposite polarity, will counterbalance the static effect; and, third, the better balancing of track currents to points of load, thereby reducing the inductive effect on adjacent communication circuits.

Inductive Effects.—The conditions producing induction in paralleling circuits must be referred to at some length in order to make the functions of the corrective devices clear. Energy is transferred through space from one

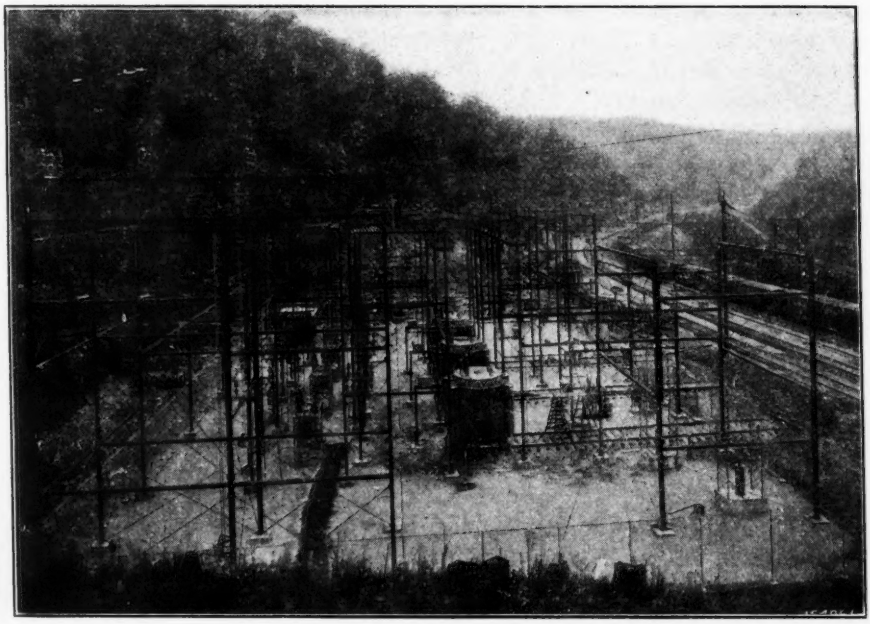


FIG. 15.—TYPICAL MAIN TRANSFORMER SUB-STATION.

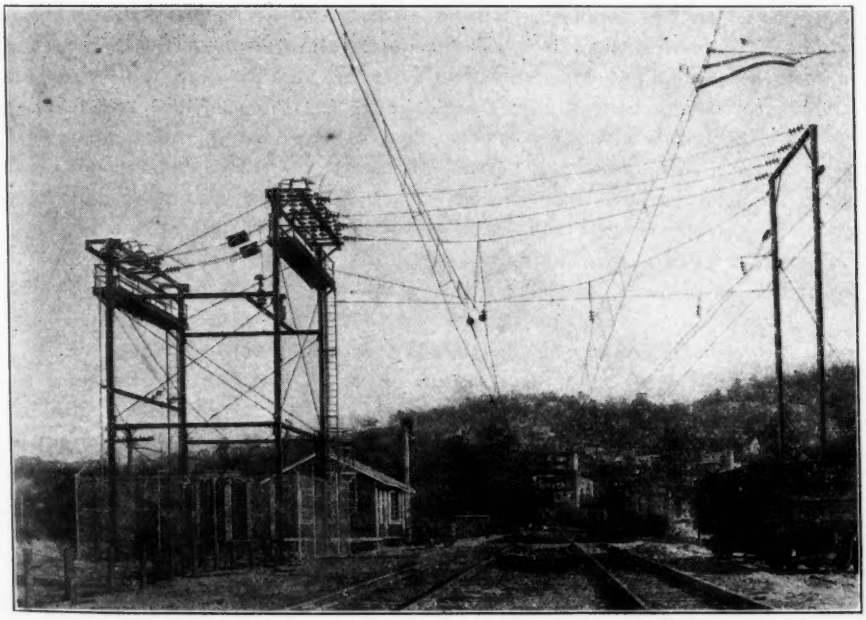


FIG. 16.—TYPICAL BALANCING TRANSFORMER SUB-STATION.

alternating-current circuit to another, due to changes in voltage or amperage of currents in either circuit, and is called "induction". The magnitude of this effect depends on a number of factors and conditions and is common to all forms of power transmission and distribution in varying degree. It is especially great under the condition of derangement of the power lines which causes grounds or short circuits. The extension of the application of powerful electric currents in transportation and for the manifold every-day uses of the public must continue, as must also the use of feeble currents in "communication" systems. Neither should be allowed to exclude the essentials for the successful application of the other, which means that correctives against interference may have to be applied in both systems to insure satisfactory operation.

Various methods, or a combination of methods, may be utilized to reduce or mitigate troubles from induction in communication circuits. Of course the most effective way is to secure considerable separation between the feeble current line and a paralleling power line. Where a sufficient separation cannot be obtained, a practical method for reducing induction from traction currents is by some form of three-wire traction distribution, of which the latest development for heavy railway work is the balancing wire system mentioned. In stating this, of course, it is understood that the communication line must be the highest modern standard and, in addition, that special devices may be required in it to obtain the fullest immunity, even when all practical correctives have been applied to the power line.

A difference of potential often exists between the trolley bus of one sub-station and the trolley buses of those adjacent, occasioned in part by unequal loading of the step-down transformers and in part by a difference in transmission voltage drop. These potential differences are communicated also to the balancing wire buses, which condition sets up a flow of equalizing current through the balancing feeder, equal and opposite to the flow through the trolley conductor. Substantially no equalizing current flows through the track rails from one sub-station to another due to this through feeding from adjacent ones. This condition, whether resulting from heavy concentration of loads or from abnormal current flow due to insulation failures, greatly reduces the inductive effect on closely paralleling communication circuits as compared with that usual with a two-wire system of distribution.

This balancing wire forms an auxiliary transmission circuit to utilize the installed capacities at adjacent sub-stations more fully than otherwise would be possible; and it effects, also, an improvement in voltage regulation. At the far end of the railway electrification it permitted the installation of an auto-transformer without the necessity of extending the high-tension transmission eleven miles to that point. The somewhat devious route of the railroad, occasioned by the topography of the country, and the wide spacing (20 miles) between main sub-stations, secured by the adoption of this balancing wire system, effected a considerable saving in length of the cross-country transmission by means of long direct-line runs. This would not have been possible with the ordinary and close sub-station spacing; it would have been

necessary to run the line nearly parallel with and close to the railway so as to tie in at the more frequent points.

Sectionalization.—Novel methods were adopted in the circuit switching for this installation. In the event of a short circuit in a trolley section supplied by two sub-stations, the breaker supplying the heavier current caused by the "short" usually opens first, leaving a stub-end short-circuit flow of current from the adjacent sub-station until the current is ruptured at that point. In the meantime severe induction would occur in a two-wire distribution system. In a balancing wire system, however, by using automatic circuit breakers only on the high-tension side of the transformers, the opening of one high-tension breaker and the bus tie breaker leaves both secondary coils of the step-down transformer still connected across the trolley and the balancing wire, which now acts as an auto-transformer, so that stub-end induction effects are avoided at all times.

High-tension breakers, actuated by directional relays, connected in both trolley and feeder conductors on either side of the tie breakers, are so arranged that only the breaker directly feeding the "short" will open, leaving the adjacent section still energized. The bus tie breakers open automatically in case of short circuit. They are also arranged to re-close automatically on the re-energizing of the line from either end, thus establishing through feed to the load from both directions.

The Catenary

The term, "catenary", refers to the form of vertical curve which the messenger cable takes when subjected to a uniform loading between supports; but in connection with an electric traction system the word is generally used to designate broadly the contact devices and supports for such systems over the railway tracks. The contact system is, perhaps, the most vitally important part of a traction installation. It is the means of getting current to the train at any place and under any conditions. As it must be adapted to widely varying physical conditions, and must function properly under all, much ingenuity has been exercised by designers in the development of an effective, durable, and sightly structure and one which can be applied at a reasonable first cost.

The Virginian installation illustrates what has been accomplished in these respects by years of experience with such work. The supporting structures are poles or columns, in single piece; brackets or cross-wires supporting the catenary wire proper; and back-guying to provide anchorage against cross-stresses. The use of high voltage in the trolley wire makes it possible to keep down the dead weight of the system by reducing the size and number of conductors to be supported. In the design of this wire, two elements are of first importance, namely, the contact wire must have sufficient current-carrying capacity to supply all demands and it must provide a sparkless collection of current. To accomplish the latter, the contact wire must be maintained in a fairly level position over the track and be supported so that it has a substantially uniform "contact quality" or "softness". Practical experience in the

design of such a system is of special value in successfully dealing with this difficult problem of sparkless current collection.

The sparking tendency at a contact may be minimized by reducing the amount (amperes) of current collected. While opinions differ as to the maximum quantity collectable, there can be no doubt that the smaller the current the better will be the collection without excessive pantograph pressure on the wire. Logically, therefore, for all reasons, the trolley current voltage should be made as high as practicable. Consequently, the efforts of designers of modern electric traction installations have been directed toward raising the trolley voltage to the highest possible value.

Amount of Current to Be Collected.—The Virginian Railway locomotives consist of three cabs, each of which is equipped with a pantograph having its sliding surface in the form of two parallel thin metal shoes. Each shoe is intended to collect its proportionate share of current. In service the locomotive is run with two pantographs in operation which results in four sliding contacts functioning on the trolley wire. The voltage in the wire is normally 11 000 and the current required at this voltage, to accelerate a train, is from 1 000 to 1 200 amperes per locomotive, or about 300 amperes per contact. During normal running about 150 amperes per contact are collected.

Experience on other railways indicates that 300 amperes can be collected from a single trolley wire with a comfortable margin for the variables occasioned by the usual grade of maintenance of a complicated wire system of the kind. However, in the case of the Virginian Railway, to provide for the possibility that in the future it may become desirable to raise the trolley voltage, the transformers and switches in the sub-stations and on the locomotives have been so constructed that the working voltage may be made 22 000 by simple and inexpensive changes in connections.

Size of Wires.—The contact and supporting wires, over the heavy demand portions of the railway were designed to carry 1 500 amperes for 10 min. on a hot day without suffering a loss of tension through over-heating sufficiently to result in a badly sagging line. On the light grade portions, the system was designed to carry 850 amperes for 10 min., under similar conditions. The distribution of this carrying capacity is as follows:

The trolley wire is a 3/0 grooved wire of copper tin bronze, drawn to a tensile strength of 60 000 lb. per sq. in. and characterized by its resistance to wear, toughness, and ability to withstand high tensions continuously without permanent stretch. Its conductivity is 65% of that of copper. Above, and parallel to, the trolley wire is an auxiliary messenger of hard drawn copper. This wire acts as an electrical feeder in addition to assisting to secure a soft contact line; finally, the main messenger (catenary cable) has considerable electrical conducting capacity. This messenger is a composite cable, having a stranded core of special composition bronze wires of 125 000 lb. per sq. in., tensile strength and surrounded by a covering of hard-drawn copper wires. No special feeder cables were required in addition to the essential wires of the catenary construction described. In a sense, however, the "balancing" wire is a feeder, but it was used for another main purpose, as has been mentioned.

Suspension Method.—The single trolley, or contact, wire is carried from an auxiliary wire by clips; this latter wire is, in turn, suspended from the main messenger by wire hangers, clipped to the auxiliary. The messenger cable is hung through a string of insulators of the suspension type to a bracket carried from a side pole, in single-track construction (Fig. 17), or from a cross-catenary wire carried by two poles, one on either side of the tracks, in cases of multiple tracks (Fig. 18). This, in general, is normal tangent construction. Special methods of suspension, however, are required to insure a "soft" contact line on curves and through overhead wire switches and section breaks.

Materials.—Reference has already been made to the non-ferrous composition of the wires. In addition, all trolley hangers and clips are of bronze. This non-ferrous wire system produces greater variations in tension, due to temperature changes, than a steel messenger cable, and, consequently, a greater variation in the height of the trolley wire. The tension of the latter, however, was made sufficiently high so that in summer it will not become flabby and thus produce an undesirable variation in contact conditions; and, on the other hand, so that tensions shall be within the elastic limit of the materials in extremely cold weather. No mechanical tensioning devices are used or required under any condition, as in a properly designed system the temperature stresses are relieved by the elasticity of the wires, by the means of carrying them, and by the elasticity of the supporting poles and cross-wires. The frequent occurrence of curves, also, materially assists in distributing stresses.

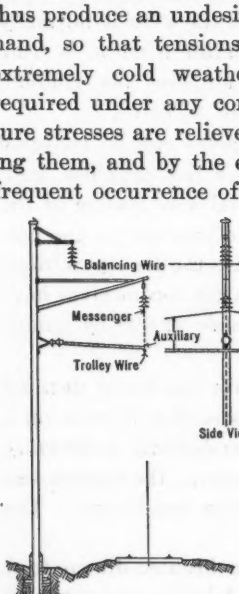


FIG. 17.—TYPICAL TANGENT BRACKET.

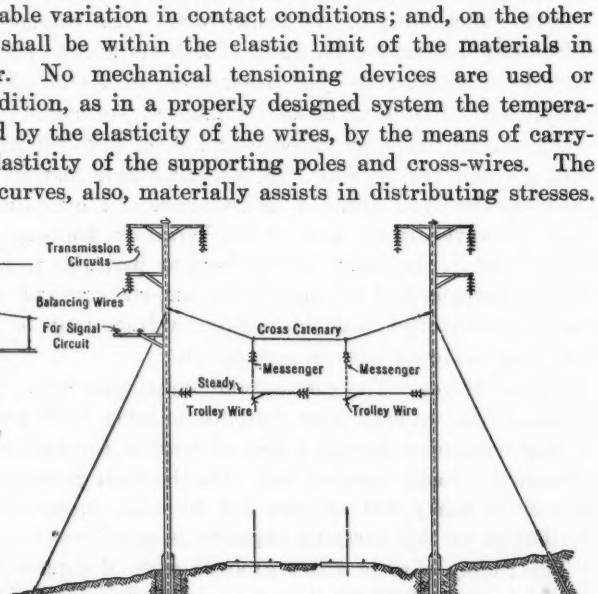


FIG. 18.—TYPICAL TANGENT CROSS-CATENARY BRIDGE.

As it is strung the trolley wire is given a tension of 3 000 lb. at a temperature of 60° Fahr. This tension is expected to increase to approximately 4 800 lb. at a temperature of 20° below zero. The messenger is given a sag of 5½ ft. in a 300-ft. span at a temperature of 60° Fahr.

Curves and Special Work.—Two methods of hanging the trolley wire from the messenger are in general use in America for curved tracks. One is the "chord" construction, in which, by a series of pull-offs, the trolley and messenger are kept approximately over the center of the track in a series of

chords. This form of construction is somewhat unsightly, as the catenary bridges or pull-off poles must be placed comparatively close together to avoid too great a departure of the wires from the track curve. Chord construction was used only in the yards on the Virginian Railway.

The other method—the one used on the main line—is the "curved catenary" construction, in which the trolley wire follows the center line of the track fairly closely around the curve. This is accomplished by the use of inclined hangers, the messenger being offset the proper amount at its supports, as predetermined by calculation. This form permits the supporting structures to be spaced farther apart than in case of chord construction, and the wire system presents a very neat appearance. Where the track curves are very sharp, "pull-offs" for the trolley wire are used in addition, to permit greater spacing of the supporting structure and to limit the incline of the hangers to an angle which will not cause interference with the pantograph on the locomotive.

The sloping hanger construction produces flexibility by allowing the wires to swing around the messenger cable as a center, and tends to eliminate "hard spots". A "hard spot" may be caused either by the increased inertia of a heavy fitting, inserted locally, or by a cusp or kink in the wire; also, by the application to the contact wire of a force (push or pull) in a vertical direction. Any of these conditions has the effect of interrupting the wave produced by an advancing pantograph, which thus may be made to rebound from the wire. This, of course, results in sparking and burning. In the design of the hardware, or fittings, it is the aim to reduce the concentrated weights as much as possible, also to avoid rigidly braced points, vertical cusps, or sudden changes of direction of the wire, and struts that directly push or bear down on the wire.

Section Breaks.—In order to divide the trolley properly into convenient electrical sections insulated one from the other, to prevent faults from being communicated to the whole system, and to isolate sections when making repairs, some form of separation must be used between adjacent sections which will permit the sparkless passage of the locomotive pantograph across the break. Wooden sticks have frequently been used for this purpose but they are not entirely satisfactory as insulators, are mechanically uncertain, and are so heavy as to cause "hard spots" in the trolley wire, which tend to cause arcs as the pantograph passes.

To avoid the use of wood and to secure a soft line, "air-gap" section breaks are employed. In these, the two sections of trolley wire are lapped past each other, separated horizontally about 12 in. At the point of transfer the wires gradually rise above the level of the contact line and their tension is finally carried back through porcelain strain insulators at a higher level into the trolley of the next electrical section. The lapped trolleys are suspended from the same messenger by hangers and are kept separated horizontally by bronze deflector rods.

Insulators.—The trolley insulators are of the suspension type and are used in strings of three. They are porcelain disks, 10 in. in diameter, cemented into malleable iron housings. Each unit has a rating of 16 000 volts and the three

together will withstand a dry flash-over test of 190 000 volts. In certain tunnels, on account of limited clearance, it was impossible to use the suspension insulators. Two-piece insulators of the rigid type, two in series, were, therefore, employed. Great care was exercised in the design of the tunnel insulation, because of the importance of insuring reliability in the presence of moisture, smoke, and gases, and because of the difficulty of making renewals under traffic.

Supporting Structures.—In general, the side supports, or poles, for the catenary are spaced from 300 to 320 ft. apart on tangents. On curves, the spans must be shorter, for reasons already given. On very sharp curves, special pull-off poles (see Fig. 19) are used in addition to the supporting poles.

For single-track, and for a portion of the two-track, line, the supporting structure is a single pole carrying a bracket (Fig. 17); the major portion of the installation is of this type. For multiple tracks a one-piece pole, on either side of the track and back-guyed, is used (Fig. 18). In this latter construction no rigid cross-member is required. Both tapered tubular poles or H-columns of a uniform section are suitable as vertical supporting members, but in case of the Virginian Railway the H-column type was employed because of economy, because it is easily climbed by the use of "climbing skates" and because wire attachments are readily made. The cross-member, which carries the main catenary system, is a wire strand, a form of construction the engineers have developed and used with success on other railway electrifications. The distinctive features of the catenary system described, are the production of a relatively cheap and elastic construction which is as light as is consistent with durability and safety. Incidentally, the design offers the least interference to vision along the tracks from the locomotive. Fig. 20 shows cross-catenary construction on yard tracks.

Poles or columns having continuous surfaces are to be preferred over latticed structure supports, not only from the standpoint of first cost, but also because they present vertical surfaces quickly drained and automatically cleaned by rain, whereas lattice-bar columns tend to collect dirt and moisture, which lead to corrosion. Repainting is also a relatively simple and cheap performance.

Foundations.—Heretofore, it has been common practice to set poles on concrete foundations poured in forms in place. In this installation a novel type of setting was devised which greatly reduced the cost of the foundations and did not require the distribution of concreting materials along the tracks, with the attendant added cost and considerable interference to traffic during construction. It was aimed to approximate the simplicity and cheapness of a wooden telegraph-pole setting. In carrying this out the columns were first served with a reinforced concrete casing on their lower ends. This was done in a yard at Roanoke some weeks in advance of distributing the columns to their sites. The design of this reinforcing and protecting casing is shown in Fig. 21.

The foundation proper was made by digging holes of comparatively small diameter in the desired locations along the track; broken stone was then placed

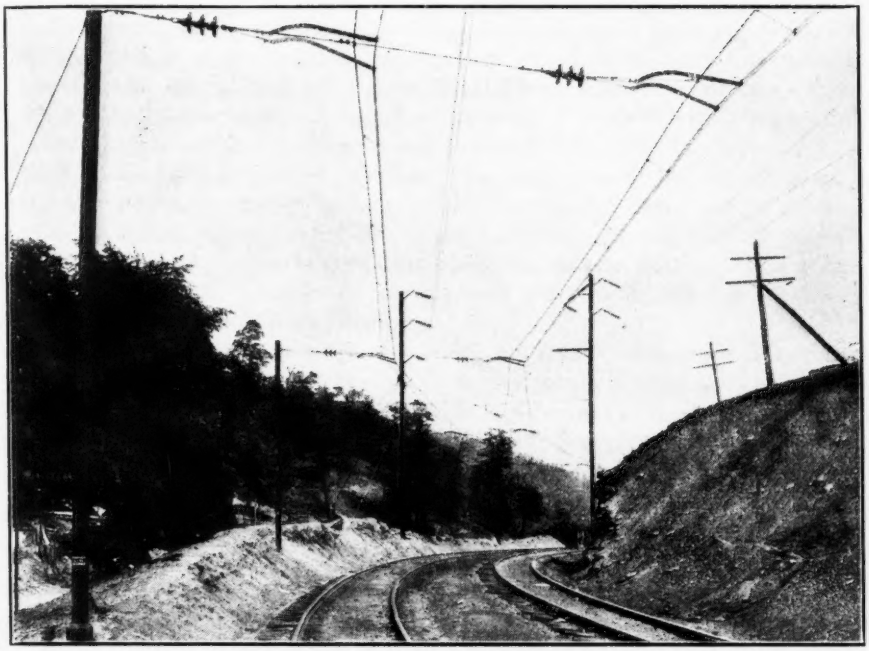


FIG. 19.—TYPICAL TWO-TRACK CATENARY CONSTRUCTION ON HIGH DEGREE CURVES.

FIG. 20.—CROSS-CATENARY CONSTRUCTION ON YARD TRACKS.

at the bottom and tamped to a predetermined elevation from the track rail. In the case of a few columns, which were designed to carry exceptionally heavy loads, cement and sand were added to the broken stone tamped into the bottom of the holes. A length of corrugated steel culvert pipe, slightly larger than the column casing, was then placed upright in the hole and back-filled and tamped around the outside. The design made two things possible. It permitted any number of holes to be opened and completed in advance without danger of caving in before the columns were set; and it provided the culvert pipe as a surface against which the column base could be wedged and left, without holding the derrick and work-train while it was being plumbed and the hole back-filled.

The cased columns were rapidly distributed from a work-train and dropped in place in the holes by a derrick. Then they were plumbed and the culverts filled with earth and broken stone, which was tamped around the casings. A surprisingly strong bond was thereby obtained between the base and the culvert pipe, as was demonstrated when there was occasion to pull up and re-locate several columns. In such cases, it was found that the column, casing, fill, and culvert pipe came out of the ground as a unit when pulled up by the derrick.

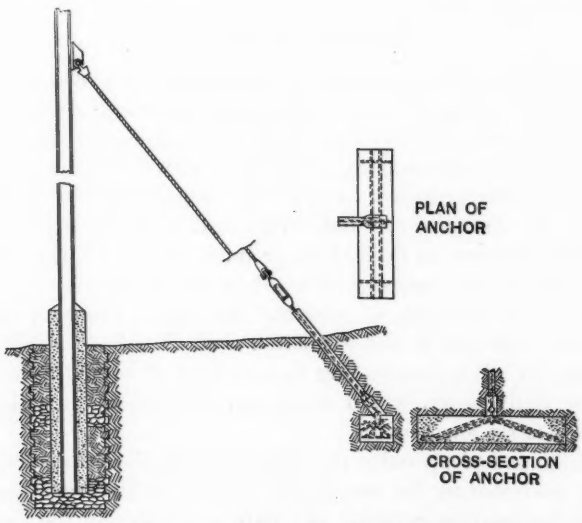


FIG. 21.—GUY AND ANCHOR FOR CATENARY SUPPORTS.

Guys.—The columns are guyed with extra high strength galvanized steel strand attached to anchors set in the ground (see Fig. 21). These anchors are pre-cast rectangular blocks of concrete, 6½ ft. long by 18 in. wide by 14 to 17 in. deep, and weigh from 1 720 to 2 100 lb. each. They are reinforced with longitudinal rods, 1 in. square, and cross-rods, ½ in. square.

The guy strand is connected to the anchor by a steel rod stub which is protected where it enters the ground by a length of steel boiler tube filled with grout. The guy is attached to the anchor by a swivel connection and to the

strand by a turnbuckle. The stresses on these guys and anchors are considerable, amounting at times to 35 000 lb., but experience covering several years (in three other installations) has shown that the construction is an entirely dependable form of pole bracing, where space permits of its use, and that it constitutes a valuable means of lightening and cheapening catenary construction. On tangents, the bracket posts are used without guys; on curves, they are back-guyed.

Track Return.—The means of returning the current from the locomotive to the sub-stations is through the bonded track rails. The running tracks are equipped with both 130 and 100-lb. rails and with lighter rails in sidings and yard tracks. All main running track rails are connected with two bonds per joint. Passing sidings have one bond only per joint; yard tracks are single-bonded, in one rail only. The bonds are No. 1/0 copper with expanded pin terminals, installed under the joint plates. Cross-bonds of one 4/0 copper are inserted at ½-mile intervals.

At the main sub-stations and balancer transformer sub-stations the track rails are connected to the buses by weather-proof insulated copper cables laid underground in trunking.

Electric Locomotives

Number Required.—The number of locomotives was determined from the train sheet of the assumed traffic, with the usual margins allowed to permit of the delays to be expected, and for flexibility in meeting business variables; also, to allow spares for shopping purposes. An initial equipment of twelve 3-cab locomotives was provided, as sufficient to perform the required schedule comfortably. As the cabs are individual and separable, this gave 36 cabs which could be grouped as desired to produce single, 2-cab or 3-cab locomotives, and obviating the necessity of withholding an excessive number of cabs from service for overhauling or repairs. To handle the assumed volume of traffic by steam traction, it was estimated that sixty-four Mallet locomotives would be required. It is interesting to note that these sixty-four steam locomotives, in working order, would weigh approximately three times as much as the twelve electric locomotives.

Mechanically and electrically the Virginian Railway locomotive embodies developments made during the past fourteen years on the Norfolk and Western Railway, and the hauling capacity per unit is practically the same as in the case of the latest electric locomotive adopted by the neighboring road. The 3-cab locomotive of the Virginian Railway represents the greatest concentration of tractive effort in one singly controlled machine found in any present-day practice, steam or electric. To secure this, advantage has been taken of the fact that an unlimited number of driving axles can be electrically actuated by control from one point, in the manner familiar in multiple-unit electric passenger-train operation. Great capacity is thus readily secured by semi-permanently connecting together separate frame units, each with its own assemblage of wheels and driving mechanism designed for proper tracking characteristics.

Starting Conditions.—In fixing on a type of locomotive, the difficulty of starting very long and heavy trains on mountain grades must be taken into account. A successful start can be made only in a very gradual manner, by a series of impulses taking up the car-coupling slack in the train progressively, moving very slowly, a few cars at one time, until all are in motion. Otherwise, train "break-in-twos" will result.

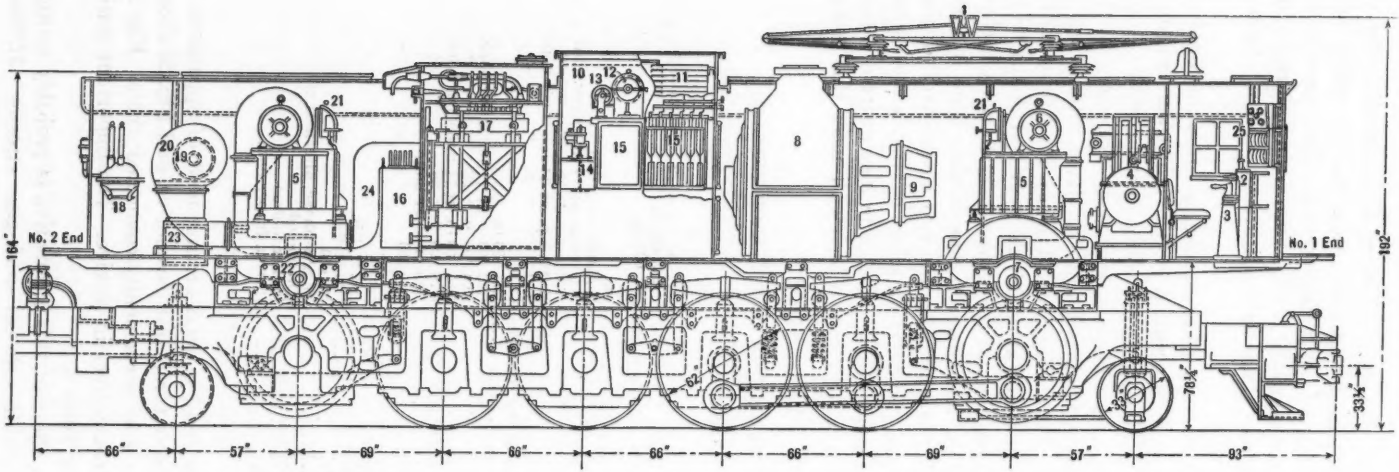
Furthermore, on the heavy grades a pusher engine is often required at the rear of the train, and it must come into action at the right time and must take up its share of the load. To insure this coincident action with a train nearly a mile long is often difficult, as whistle signals may not carry to the rear, especially in crooked and narrow wooded valleys. It is interesting to note that on the westerly portion of the Virginian Railway for twenty miles on the heavy grade, there is only one point at which the rear end of the train can be seen from its head. It is, therefore, at times necessary to slack back to give a "bump" signal to the pusher, and often a considerable time interval may elapse in obtaining concerted action between the two locomotives, during which period the one at the rear may be exerting effort against a motionless train. Under these conditions the motors are subjected to high-current inputs and may be damaged unless they are of a type which will meet this severe requirement.

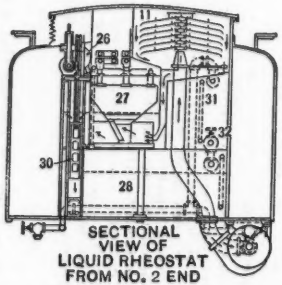
TABLE 5.—Characteristics of the Locomotives.

Number of three-unit locomotives	12
Number of cabs, or units each	3
Classification as to wheel arrangement	(2 − 8 − 2) + (2 − 8 − 2) + (2 − 8 − 2)
Total weight, approximately	1 285 000 lb.
Total weight on drivers, approximately	937 000 lb.
Number of driving axles	12
Number of motors	6
Type	3-phase induction
Total capacity, at 1-hour rating	6 000 h.p. at 14 miles per hour 6 990 " " 28 " " "
Total capacity, continuous rating	5 000 h.p. at 14 miles per hour 6 000 " " 28 " " "
Maximum starting tractive effort (as limited by electrical capacity equivalent to 30% adhesion)	277 500 lb.
Tractive effort, at 1-hour rating	162 000 " at 14 miles per hour 94 500 " " 28 " " "
Tractive effort, at continuous rating	135 000 lb. at 14 miles per hour 78 800 " " 28 " " "
Maximum safe speed	38 miles per hour
Gear ratio	21 : 100
Diameter of driving wheels	62 in.
" " pony-truck wheels	33 "
Length over-all (pulling faces of couplers)	152 ft. 3 in.
Rigid wheel-base (each unit)	16 ft. 6 in.
Voltage of trolley current	11 000
Kind of current	Single-phase, alternating
Transformer	Oil-cooled
Phase converter	Synchronous
Air-brake compressor capacity (each unit)	150 cu. ft. per min.

Type.—Fig. 22 shows the wheel and general arrangement of apparatus of one of the units making up a complete three-unit locomotive, and Fig. 23 shows the manner of grouping units in a semi-permanently connected whole. Fig. 24 is a view of one of these electric locomotives in service. The important statistics of the locomotive are given in Table 5.

Tractive Effort.—The tractive effort of this locomotive is probably as high as desirable, in view of keeping within safe car draft-gear stresses. If, how-





1. Pantograph
2. Master Controller
3. Air Brake Valve Pedestal
4. Air Compressor
5. Control Apparatus
6. Traction Motor Blower
7. Traction Motor No. 1
8. Phase Converter
9. Phase Converter Motor
10. Liquid Rheostat
11. Cooling Tower
12. Valve Operating Mechanism
13. Interlock Drum
14. Auxiliary Dump Valve
15. Electrode Compartments
16. Preventive Coil
17. Main Transformer
18. Oil Circuit Breaker
19. Transformer Oil Pump Motor
20. Blower
21. Blower Motor Switch
22. Traction Motor No. 2
23. Radiator for Transformer
24. Air Duct for Rheo. & Prev. Coil
25. Air Gauges & Ammeters
26. Operating Motor
27. Electrode Compartment
28. Electrolyte Tank
29. Rheostat Circulating Pump
30. Main Overflow Valve
31. Fresh Water Storage Tank
32. Electrolyte Tank Filling Valve

FIG. 22.—DIAGRAM OF ONE OF THREE-UNIT ELECTRIC LOCOMOTIVES, SHOWING APPARATUS, VIRGINIAN RAILWAY.

ever, it should be demonstrated in the future that longer and heavier trains can be handled reliably, the tractive effort of the locomotive may be readily increased one-third by adding one more unit. Provisions for this contingency have been made not only in the locomotive, but in the power system throughout.

Drive.—The running gear of each cab comprises four driving and two truck axles (Fig. 23). The drive is through side rods from the jack-shafts, each of which is connected to two pairs of wheels by main and side rods. These two driving couples are not interconnected because it was regarded better practice to avoid excessive rigidity of the system. The rod motion is a purely rotary one and, therefore, can be perfectly balanced by counterweights, thus eliminating the effect of dynamic augment at the track, such as necessarily results from the unbalanced reciprocating parts of the steam locomotive. The jack-shaft gears have spring-cushioned rims, one gear being mounted on each end of the jack-shaft.

The use of a rod drive is advantageous, in allowing maximum length of motor armatures, and hence maximum capacity, also in giving a simple form of connection between parts of the mechanism having relative vertical play, due to the wheel equalization and the springs. Furthermore, coupling of the axles by side rods insures maximum total adhesion for the locomotive as it avoids diminution of tractive effort which occurs with separately geared axles, due to inequality of weight distribution and to weight transfer at low-track rail joints, etc.

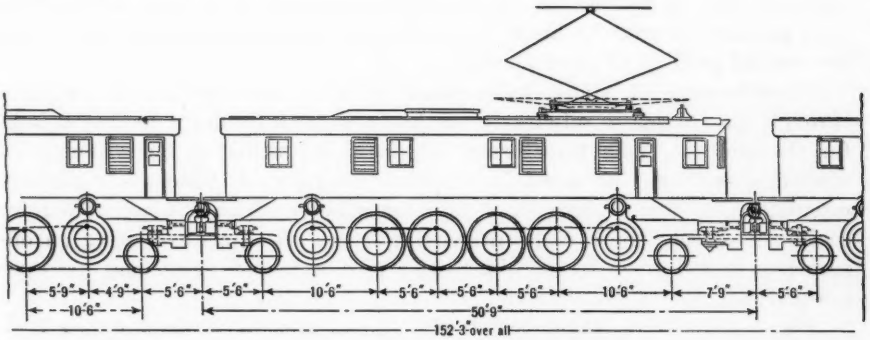


FIG. 23.—GENERAL DATA FOR THREE-UNIT ELECTRIC LOCOMOTIVE, VIRGINIAN RAILWAY.

Motors.—Each locomotive unit has two motors (Fig. 22), one set near each end of its frame and geared to a jack-shaft mounted directly beneath. The motors are 3-phase, having non-variable speed characteristics, that is, they operate at certain set speeds, irrespective of the loading and determined by the combination of motor-pole connections set up. Two standard running speeds were adopted, namely, 14 and 28 miles per hour. The former is used on the heavy grades; the latter on the level and light grades.

It has been found on the Norfolk and Western Railway, in similar service, that these speeds admirably suit the haulage of heavy "drag" freight trains, insuring the maintenance of schedule without excessive speeds over any

portion of the run. In certain kinds of railway service one inflexible speed characteristic is objectionable, but in "drag" service where every train is of maximum weight it is advantageous, as the maintenance of absolutely uniform speed, regardless of change of grade, tends to prevent "break-in-twos" due to acceleration and retardation locally within the train itself.

Furthermore, the proper running speed should be accurately gauged and maintained over a road with the grades and curvature found on the Virginian Railway (Fig. 25). When regenerating down grade, the speeds obtainable are about 15 and 30 miles per hour. The change from motoring to regenerating, or *vice versa,* is automatic as the grade changes, without the master controller being thrown to the "off" position. The only action required of the engineman to accomplish the change is a movement of the transformer balancing switch lever.

The three-phase motor is especially well adapted to the difficult train-starting conditions, referred to previously. It is simple and robust in the construction of its revolving element and has no commutator; it is also of large output capacity for a given weight and dimensions.

Current is furnished from the contact wire to the locomotive and controlled through the main electric apparatus. The arrangement and relation of these various parts are shown in Figs. 22, 23, and 24.

Pantograph.—This is a lazy-tong contact device set on the roof of the locomotive cab. It collects current at 11 000 volts from the trolley wire. The mechanism has latitude of motion vertically for the variable wire heights, and laterally the pantograph contact surface is made as wide as tunnel clearance permits, in order to insure contact where the trolley wire lies away from the central position of the contact pan.

Transformer.—A static transformer in each cab receives the current through an automatic oil-circuit breaker and steps it down to low voltage for the motors. This transformer is of the oil-insulated type, having its windings and core immersed in a tank containing oil which is constantly circulating by pumps through a radiator against which a blast of air for cooling is directed by a motor-driven fan. The advantages of using an oil-insulated transformer are considerable where coal dust, snow, and moisture are present.

Phase Converter.—Inasmuch as the trolley current is single-phase it is necessary to provide apparatus on the locomotive to convert it to three-phase for use in the induction motors, hence the term, "split-phase", is sometimes used to describe this type of locomotive. This conversion is effected by means of a rotary "phase-converter", which consists of a synchronous motor with a two-phase wound stator and a wound rotor. One motor stator phase is fed from the transformer and proportional voltage is generated in the other of a phase displaced approximately 90° electrically from the first. One terminal of the generated phase is connected to the mid-point of the transformer secondary, resulting in a "Scott" connection to give three-phase power. The power factor of this three-phase circuit is high (about 95%) by reason of the synchronous type converter used. In an emergency the converter may be operated as a straight induction type.

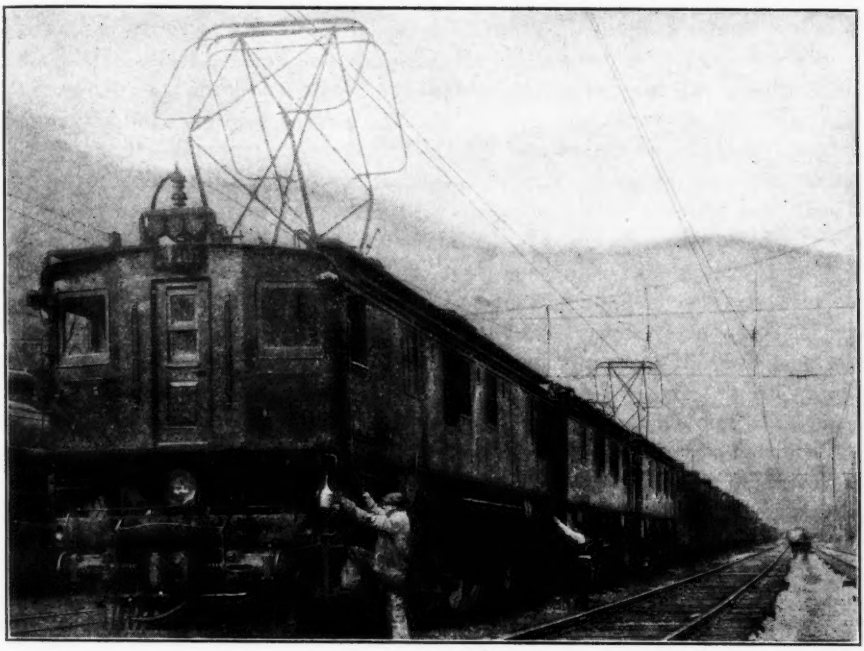


FIG. 24.—ELECTRIC LOCOMOTIVE, VIRGINIAN RAILWAY.

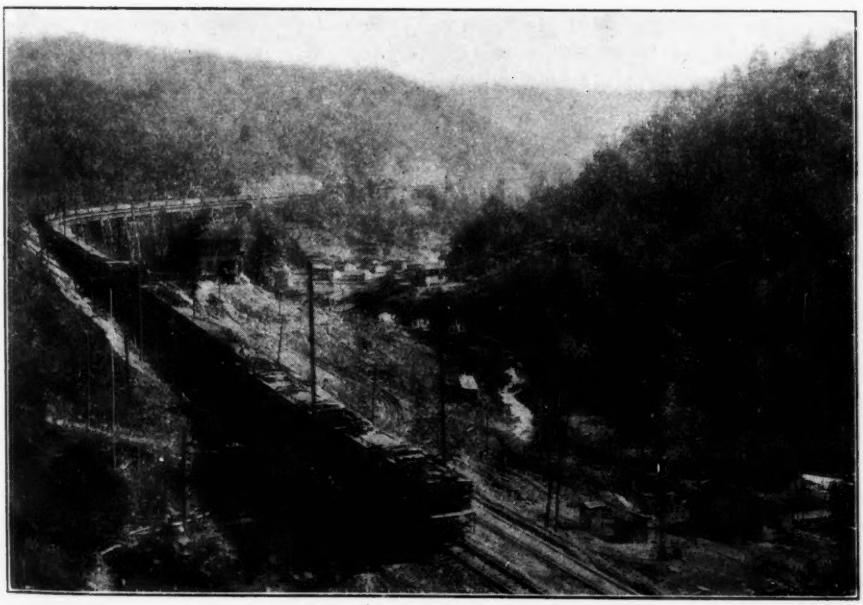


FIG. 25.—EMPTY TRAIN DESCENDING MOUNTAIN GRADE NEAR WEST END OF LINE.

The converter is started and brought up to synchronous speed by a commutating motor of the single-phase series type, mounted on an extension of the shaft. After synchronous speed is reached this motor is automatically reconnected as a direct-current generator to supply current for exciting the rotor windings of the converter.

Rheostat.—Gradual acceleration of the locomotive to running speed is effected by inserting a variable resistance in the secondary, or rotor, circuit of the traction motors by means of a rheostat. This rheostat can also be used to vary the locomotive tractive effort and speed when required.

The rheostat is of the electrolytic type, contained in a tank. It consists of metallic electrodes immersed in an unsaturated aqueous solution of sodic-carbonate. This electrolyte is controlled by an overflow valve which may be raised or lowered by a motor-driven mechanism to vary the height of the liquid around the electrodes and thus to vary the resistance in the motor circuit. As the electrolyte absorbs energy and thus becomes hot, it is circulated by a pump through a cooling tower. In addition to the main overflow valve, which regulates the height of the liquid, two auxiliary valves are provided, one for each electrode compartment. Either of these valves can be opened by a button in the motorman's controller to lower the level of the electrolyte rapidly in the compartment it controls, independently of the position of the main overflow valve. This means is used to correct the slipping of one set of drivers, or to reset the overload relays of one motor without effecting the tractive effort of the other.

Mechanical Construction.—The main backbone of each locomotive unit consists of two cast vanadium steel side-frames, each in one piece, 42 ft. long, and tied together by end and intermediate castings which form motor supports and cross-ties. Between these ties a cast-steel bed is provided to form a continuous support for the heavy electrical apparatus, such as the phase converter, liquid rheostat, transformer, air compressor, etc., mounted on it. The whole frame assemblage is unusually strong in order to resist bumping and pulling shocks and any abnormal strains such as would be imposed if the locomotive should be lifted with the pedestal binders of the driving axle removed.

The frames and bed-plate carry all the driving mechanism and apparatus independently of the cab, or housing, mounted on it. This arrangement also provides means for supporting the cab rigidly on the substructure without the use of intermediary springs, and makes the cab, in effect, merely a box or shell to cover the apparatus and to carry the pantograph. Hatches are provided in the roof so that all apparatus may be lifted out by the shop traveling crane.

At each end of the frame is a two-wheeled pony truck. It is of the radius bar type with special suspension designed to give the desired resistance characteristics for tracking around curves and to distribute wheel-flange pressures properly between the truck and drivers.

No effort has been made to lighten the mechanical parts in order to obtain high weight efficiency of the locomotive as a whole. It was regarded

of importance, rather, to obtain ruggedness and durability in an extremely severe service.

For ready reference, a short summary of construction and operating data is given in Table 6.

TABLE 6.—CONDENSED DATA.

Dates	Engineering started	June 1, 1923
	Construction started	November 29, 1923
	Regular electric operation commenced between Mullens and Princeton	September 24, 1925
	Regular electric operation commenced over entire Division, Mullens to Roanoke	September 18, 1926
	Note.—Construction work suspended east of Princeton for three months, October to December, 1925, and resumed in January, 1926.	
Right of way	Route miles electrified	134
	Track " "	231
	Maximum grades, percentage	2
	Maximum curvature, degrees	12
	Percentage of line on curves of more than 2 degrees	54
Capacity	Tonnage of coal eastbound for which electrification was designed:	
	Per year	12 500 000
	Per maximum day	57 000
Train weight	Locomotives, cars, and lading on Clarks Gap Hill (two locomotives), in tons	7 285
	East of Princeton (one locomotive), in tons	9 640
	Westbound empty movement (one locomotive), in tons	3 440
Train speeds, running	Percentage of run at 14 miles per hour	53
	Percentage of run at 28 miles per hour	47
Maximum number of trains in motion and taking power	Simultaneous, eastbound	5
	" westbound	2
	Total taking power at one time	7
	Total number of loaded cars moved eastbound on heavy day	2 200
	Total tonnage moved per heavy day eastbound and westbound (gross)	95 400
	Ton-miles (trailing) per month	160 000 000
Power demand	Starting 7 285-ton train, accelerating to 14 miles per hour on 2% grade, at train, in kilowatts	14 000
	Starting 9 640-ton train, accelerating to 28 miles per hour on 0.2% grade, at train, in kilowatts	11 000
Regeneration	Power, in kilowatts, regenerated down hill by 9 640-ton train	5 000
	Power regenerated in percentage of total consumed in an eastbound run over the Division	20
	Power regenerated in percentage of total power-house output	8

Power-house load		Normal day.	Congested day.
	Average load, 24 hours, in kilowatts	14 500	20 500
	2-hour peaks, in kilowatts	22 300	29 200
	1-hour peaks, in kilowatts	31 300	31 500
	5-min. peaks, in kilowatts	38 300	43 500
	Total yearly power station output on basis of 12 500 000 tons of coal hauled eastbound, in kilowatt hours	112 000 000	

INSPECTION AND REPAIR FACILITIES

Locomotive inspection facilities are required at the end of a run. At a turn-around point—such as at Roanoke—where it is undesirable to hold locomotives out of service for a considerable length of time, only minor routine inspection is done. The provisions made for this are: A track pit with housing over it, sanding facilities, and a supply of water for refilling the locomotive control rheostats. All these facilities are conveniently located for quick access by the locomotives.

The main Repair Shops for locomotives and cars and the Motive Power Department headquarters are at Princeton, 37 miles east of the Division terminus. These shops continue to be required for general equipment repairs for the road, other than for the electrified portion. It was deemed inexpedient to depend on the Princeton plant for all electric locomotive overhauling and repairs, as it was inconveniently located in reference to the end of the electric locomotive runs and the point where they are held for service. Therefore, it was decided to use the Princeton Shops for heavy and class repairs only of the electric equipment, and to build at Mullens a modern and well-equipped plant for general overhauling, minor repairs, and thorough inspection (Fig. 26).

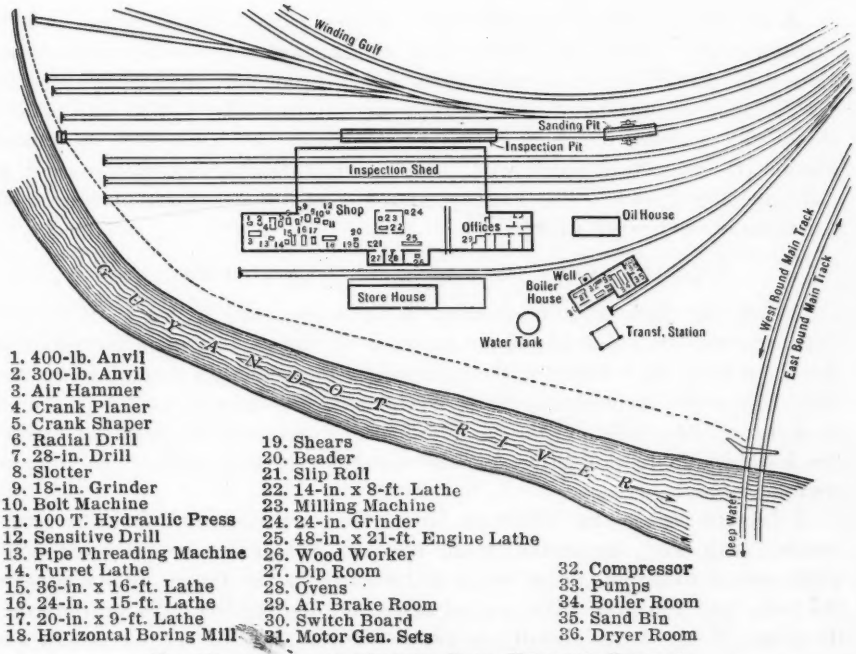


FIG. 26.—LAYOUT OF MULLENS SHOP, VIRGINIAN RAILWAY.

Mullens Shop.—The main building comprises a machine shop and an inspection track bay. It is a substantial steel framed structure, with curtain-walls of brick. The machine shop is 316 ft. long by 40 ft. wide. The front 50 ft. is fitted for offices, with a mezzanine for light machine work overhead. The rear 60 ft. houses the blacksmith shop, a foundry, and a welding shop.

A 15-ton traveling crane (hand-operated from the floor) spans the shop and travels the full length. Alongside the machine shop is a lean-to housing a carpenter shop, a dip-room, and two ovens for the impregnating and baking of motor coils. The ovens are electrically heated and are provided with automatic temperature recorders and control. The inspection shed part of the building is 206 ft. long by 70 ft. wide and covers three tracks, each long

enough to take locomotive cabs, with ample working space between each. An inspection pit is provided the entire length of each track, with a cross-pit at each end for dropping wheels; these cross-pits are equipped with hydraulic jacks. A 30-ton electrically driven traveling crane with a 10-ton auxiliary hoist spans the main shed and travels the full length.

The shop is heated by steam, supplied from a separate boiler and power plant. Electric shop power is obtained from the traction lines and is converted in this plant by a small transformer and motor-generator set from single-phase alternating current to 250 volts direct current for the shop machines. The building also contains electrically driven air compressors and service and fire pumps. The water supply is from wells and is pumped into a 125 000-gal. steel tank mounted on a tower.

A modern equipment of individually driven machine tools is provided, including engine lathes of various sizes, milling machine, boring mill, slotter, shaper, planer, drills, wood-working machine, air hammer, and blacksmith tools.

The general storehouse is in a separate building adjacent to the machine shop, with a supply track in between. Alongside the main shed an outdoor locomotive inspection track with a pit is provided for locomotives not requiring shop service. The location of many of these various buildings and machines is indicated in Fig. 26.

Railway Company Telephone and Telegraph Protection

As already explained, the current in the trolley and feeder wires may induce currents in adjacent communication circuits. The principal effect of this is to raise the voltage of the communication line with respect to ground, with a possible break-down of its equipment, or electric or acoustic shock to operators. In addition, the communication lines must be protected against fire hazard due to possibility of high-tension and communication lines becoming crossed.

Telegraph Line.—In telegraph lines, the earth is used for the return current path, and, consequently, an induced voltage in the telegraph wire, with respect to ground, may cause disturbance in the proper functioning of the relay and interfere with normal telegraph dispatching. To prevent this, therefore, in the local circuits, a low-resistance relay of special design was introduced. This has a split circuit so arranged that the 25-cycle current passes through both parts in equal amounts and hence does not operate the relay or sounder. To protect the operator against coming in contact with high induced voltages, this special relay is enclosed in a wooden box, and insulated knobs are brought through the cover. Furthermore, an insulated telegraph key is provided.

On the through circuits, resonant shunts are introduced around standard relays in order to reduce the voltage to ground when the telegraph key is open. This shunt by-passes the greater part of the 25-cycle current, so that the operation of the relay is not affected. Anti-resonant shunts, or 25-cycle chokes are also installed in all circuits to limit the flow of current to ground.

There are no protective devices used outside the zone of electric train operation, and hence means had to be provided to confine the induction to

the electrified section of the road. This was accomplished by installing special choke coils and resonant drainage coils at each end of the electrified zone, which drain off the 25-cycle current to ground. This arrangement, while it prevents normal induction from going beyond the electrified zone, does not afford protection in case of short circuits; it is, therefore, supplemented by low-voltage lightning protectors, which are designed to drain off any excess voltage.

Telephone Lines.—In the ungrounded telephone circuits the induction is about equal in both wires, and hence little disturbance is caused by normal operation. However, to provide against break-down of protection, and possible shocks to users (at times of short circuit), and also against possible crosses of the open-wire telephone lines with high-tension power wires, insulating transformers are provided on the line side of all telephone instruments in the electrified zone. These protective measures have proved effective and sufficient for good service.

Design and Construction

It is obvious from the foregoing description that, in carrying out this extensive project, a co-ordination of design in several different branches of engineering—civil, mechanical, and electrical—is involved. Furthermore, as it is necessary that design and construction shall proceed simultaneously, a large organization is needed.

The Railway Company decided, having these facts in mind, to employ the writer's firm to carry out the project, this organization to become for the time being a special department of the road. Thus, the Company would receive the benefit of a staff of designers and constructors skilled in this kind of work, and the entire installation would become a first-hand railway company undertaking. The engineers reported to the Chief Engineer of the Railway Company as regards the general approval of plans and expenditures. Purchases and auditing of accounts were also handled through the regular railway departments in the usual manner. Thus, a close contact of interested officials with the work in progress was maintained without burdening them with a multitude of detail.

Designing.—A heavy electric system cannot be applied to an existing property without a considerable number of changes in the steam railroad facilities, and, generally, a modification of certain operating methods is required to produce best results. As a basis, therefore, for preliminary information, the Company's records and plats of alignment and grade were used, and many conferences with the officials were had in order to determine the electric train sheets, and the additional operating facilities desired.

The roadway cross-sections, the tracks, tunnels, and viaducts were carefully studied, as well as the buildings and other facilities, which might affect the type and clearances of the catenary structures. All were separately checked in much detail, and supplementary surveying work was required both at the start and as designing progressed. Notes were also made of the nature of the sub-surface materials as an aid in determining the depth and character of column and guy-anchor foundations for the catenary spans. As

the transmission lines were to be run through a mountainous and heavily wooded country, extensive surveys were required, and protracted negotiations for the purchase of property or easements had to be conducted.

In order to deal with all designing problems promptly, a field organization, under the direction of a resident engineer, was formed and directed from a centrally located headquarters on the line. This official had charge of all surveying parties, and later, when construction was started, he organized and carried out the work, reporting to the home office in New York.

Many special problems in producing the designs have been mentioned. The design of the power house in itself involved much detail, but the plans for the electrical distribution system for 137 miles of railway, required the largest number of designers. Practically each catenary span had to be dealt with as a separate problem, because of the continually differing local situations as to foundations, track curvature, number of tracks, clearances, etc., all of which produce varying stress conditions. In the aggregate more than 3 000 drawings were made for the entire installation.

An important consideration in preparing designs is to reduce the amount of field cutting and fittings of parts of the catenary system during the process of erection. This was accomplished by securing accurate preliminary information as to track alignment and elevations, and by careful laying out and calculation in the drafting-room, notably of the curve and overhead special work, such as the turn-outs and section breaks. Schedules and tabular information were made up and furnished the field forces for each catenary span, including cable sags and hanger lengths, prior to the distribution of materials on the job. In this way a surprising amount of time and expense was saved in the actual erection work.

Construction.—Space admits the mention of only a few of the methods followed, principally those involved in the production of the transmission and distribution. Aside from the main construction headquarters at the Narrows Power Plant, secondary headquarters were established at several points along the railway. These were generally in camp trains, which furnished mobile living accommodations for the men. As many as five of these trains were used at times in the different operations. Certain of them carried tower cars which were used in the erection of the catenary wires and fittings. Tent camps were provided, especially for the surveying and erection forces on the transmission line, where this work was remote from the railway.

The entire work was so organized as to require the purchase of a limited amount only of special equipment to supplement the Railway Company's work-train and other construction or service apparatus and tools. A locomotive crane with a 50-ft. boom was used in erecting catenary poles, setting concrete anchors, etc. Extensive grading, where required at the sub-stations, was let to local contractors who had the necessary equipment.

The tower cars were built in the Company's shops by erecting wooden frames with extension tops on standard flat cars. A number of special automobile gasoline motor cars were used to haul air compressors for field

drilling and for the use of the adjusting and tensioning gangs; also for inspection and the transport of men and light materials. These cars were light and could be quickly removed from the track, to accommodate traffic. The aim was always to interfere as little as might be with regular train operation, with due regard, however, to a fair utilization of the time of men engaged in construction operations.

Transmission Line Erection.—The tower foundations were grillages with foot sections set in earth excavations and back-filled ready for setting up the superstructure. All this part of the work was laid out so as to require the minimum amount of handling of heavy pieces over a difficult country. The erection of tower sections was by means of a gin-pole carrying a block and tackle. Horse teams were used for haulage and for hoisting operations.

The stranded wire cable for the transmission circuits was delivered on reels, each containing about 2 600 ft. and strung through snatch-blocks attached to the tower cross-arms. The cable was afterward sagged, as called for by the engineering schedule, using block and tackle. The determination of the proper sag was by sighting through targets on the towers and on the cables at mid-span.

The rate of progress in this work was about one tower erected per day. Transmission cable was strung and sagged at the rate of 6 miles of cable per day.

Catenary Erection.—In conducting this operation, experience is especially valuable; the work must be done "under traffic", much is special to its location, and the adjustments required are many and precise.

As heretofore mentioned, the pole foundations are of a character that is not only economical in material and labor, but eliminates the necessity of requiring concreting work trains on the line and the consequent intereference with revenue trains. The foundation holes were dug, or blasted out, as the case might be, fitted with corrugated steel casings, back-filled, and capped, ready for quickly placing the poles. These were placed in position by a locomotive crane attached to the work-train which rapidly carried a number of poles. In this way frequently as many as 35 poles were erected in a 10-hour day. The guy anchors were delivered and set in the same manner. At the time of placing the poles in the foundations, they were wedged plumb by wooden blocks, which were subsequently removed when the casings were back-filled. After an anchor and pole was set it was unnecessary to use the crane to complete the erection of guys or fittings.

The various wires of the catenary system were strung in the following order, by use of extension-top tower cars in a work-train: First the balancing transmission wire, then the cross-spans, the messenger cable, and, lastly, the trolley and auxiliary wires. Proper tension was given, in accordance with the drawings, by the use of a dynamometer installed at varying distances, depending on the track alignment. An average of four reels of messenger cable each containing 4 500 ft., were strung and sagged per day; this amount, however, varied considerably, depending on traffic conditions.

Next, a separate gang distributed and erected the hangers, which had been previously cut to the lengths required by the tabulations of the designers and had been equipped with top and bottom fittings. The degree of accuracy of the designing calculations was such that little field cutting was required. The hangers were applied to the messenger cable by an employee riding the messenger in a "boatswain's chair". Subsequent operations were as follows: The erection of insulator strings, the clipping-in of auxiliary and contact wires, and the final adjustments. All these were done from light tower cars which could be quickly removed from the track, when required.

At points where special work, such as over-head switches, section breaks, deflectors, and pull-offs, occurred the operations were more complicated, and progress was slow.

Apparatus and Materials Used

All equipment was selected with due consideration of the service requirements and suitability. The electric apparatus throughout was of Westinghouse Electric and Manufacturing Company design and make. The mechanical parts of the locomotives were furnished by the American Locomotive Company. The powdered fuel combustion system is known as the "Lopulco" and was furnished by the Combustion Engineering Corporation.

Operating Results and Economies

The Virginian Railway has now (January, 1928) had full electric operation between Mullens and Roanoke for more than a year and on the section between Mullens and Princeton for approximately two years. The actual performance under electric traction has fully met expectations and the results have been satisfactory from the operating standpoint. The locomotives and the power system throughout have met the severe requirements of service. Trains of 9 000-tons trailing load are regularly operated over the line at 28 miles per hour on the low-grade sections and 14 miles per hour on the heavier grades, and are held by regenerative braking on descending grades without the use of air-brakes. The time of run between Mullens and Roanoke (134 miles), with full tonnage, as compared with steam, has been reduced 7 hours per trip, as anticipated.

The effect of electrification in increasing the capacity of the railroad, especially on the heavy grade section between Elmore and Clark's Gap, has been so pronounced that it is not anticipated that the ultimate capacity of the single-track railway will be reached, even with continued increase in business, for a long time to come.

It would be of much interest to present figures for the relative cost of operation, steam and electric, and to deduce the savings effected by the changed form of motive power, but the writer believes definite figures must follow very careful analysis covering a considerable period of time, and the conclusions cannot be safely used as a generalization in favor of electric traction elsewhere; they must be understood to apply only to the character of conditions, physical and operating, of the road in question.

It can now be safely stated that, as concerns the Virginian Railway Company's installation, savings have been very large and establish the fact that they will suffice to pay a very satisfactory return on the cost of the project, aside from a consideration of the indirect advantages secured. This result is largely due to the peculiar conditions of the service, that is, very heavy trains on a long division, operated at relatively high speed, such that the time of enginemen and train crews is greatly reduced over what is possible in a congested and slow steam operation.

Speaking in very general terms only, the main savings in operating costs on the electrified division of this road comprised reductions in three very large items, resulting, on the basis of equal gross tonnage moved, as follows:

Engine- and train-men	One-third that of steam.
Locomotive repairs	One-third that of steam.
Fuel	One-half that of steam.

Engine-house expenses, which is a comparatively small item, were reduced to one-quarter the amount required by steam service.

Personnel

This electrification was under the general direction of H. Fernstrom, M. Am. Soc. C. E., Chief Engineer of the Virginian Railway (now retired); Hugh Pattison, M. Am. Soc. C. E., Engineer of Electric Traction; John W. Sasser, Superintendent of Motive Power; and J. E. Sharpley, Electrical Engineer. The engineering, detail designing, and the construction work was carried out by Gibbs and Hill, Consulting Engineers, New York, in whose organization, in addition to the writer and E. R. Hill, M. Am. Soc. C. E., directly in charge of the designing and construction work, were Messrs. S. A. Spalding, Chief Engineer; Norman Litchfield, Mechanical Engineer; H. S. Richmond, Structural Engineer, and H. W. Hudson, M. Am. Soc. C. E., Resident Engineer, in the field.

Discussion on Page 1295

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ANALYSIS OF ARCH DAMS BY THE TRIAL LOAD METHOD

By C. H. Howell,* M. Am. Soc. C. E., and the Late A. C. Jaquith,† Esq.

Synopsis

This paper presents a method of arch dam analysis that has been gradually developed in the Designing Office of the United States Bureau of Reclamation. It is now also used by the Salt River Valley Water Users Association, the Middle Rio Grande Conservancy District, and by one of the writers for a high arch dam in Mexico. It is based on the assumption that an arch dam is composed of systems of horizontal arches and vertical cantilevers, that each system carries a proportion of the total load, and that the deflections of the two systems are equal.

The method consists in applying a series of assumed proportional loads to each system and computing their deflections. When a proportionate loading has been found that gives equal deflections at all points for both systems, the stresses are computed and are considered to be those which will obtain in the dam.

The mathematical treatment of the problem has been gradually developed and improved. The final form is given in the Appendix. If applied to the earlier designs, the numerical results would be slightly different, but the stresses and the shape of the loads would not be greatly changed.

It is now generally conceded that the cylinder formula does not give correct results when applied to arch dams. It makes no allowance for temperature changes, rib-shortening, or the effect of cantilever action. The theory is wrong and the resultant designs are uneconomical. The lack of economy increases very rapidly as the size of the dam is increased.

Note.—Written discussion on this paper will be closed in May, 1928.

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† Asst. to Chf. Engr., J. G. White Co., City of Mexico, Mexico. Mr. Jaquith died September 30, 1927.

No dam, designed as a cylinder, has failed as far as is known to the writers. However, it is probable that some of them are stressed nearly to the safe limit. Existing dams designed as cylinders are of moderate height, and the maximum allowable stresses assumed in their designs are relatively low. These facts probably account for the lack of failures.

Many arch dams of great height are now under consideration. Consequently, the development of a complete theory of design is greatly needed. This paper is not presented as a complete solution of the problem, but a step toward it.

A somewhat generally accepted method of investigating arch dams is to consider that the water load is carried by two systems of elements. One system is vertical and is considered as a series of cantilevers. The other system is horizontal and is considered as a series of arches. The two systems have equal deflections at all points and carry proportional parts of the load. It is probable that they are not exactly at right angles to each other, especially near the abutments. This complicates the problem. The amount of error caused by assuming the systems normal to each other is not known.

Several methods have been proposed for determining the distribution of the load between the arches and the cantilevers. These methods generally have been based on the relative rigidity of the arches and the cantilever at the crown. Each arch element is assumed to carry a uniform load such that, when the remainder of the unit water load is placed on the crown cantilever, the computed arch deflections at the crown will equal the computed crown cantilever deflections. It is believed that such a procedure is entirely inadequate.

Consider a canyon section, as in Fig. 1 (*a*). The cantilevers at the crown and near the abutments, being of the same height and cross-section (assuming arch elements of uniform thickness), will theoretically deflect the same amounts under equal loads. Under a uniform load, the arch deflection at the crown will be much greater than that near the abutments. Hence, if the crown deflection of the arch equals the cantilever deflection at that point, it cannot be equal to that of the cantilevers at other points. Therefore, no uniform division of load between the two systems can produce equal deflections at all points.

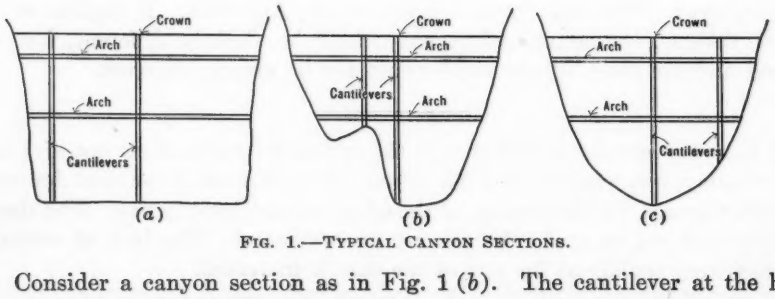

FIG. 1.—TYPICAL CANYON SECTIONS.

Consider a canyon section as in Fig. 1 (*b*). The cantilever at the hump, due to its greater stiffness, will clearly carry more of the load than the cantilever at the crown or maximum section. Therefore, the arch will carry less at the hump than at other points. In fact, the arch will undoubtedly be sub-

ject to a negative load at such a point. Some dams exist that have approximately this profile. Even in regular profiles similar to Fig. 1 (*c*) the arch loads were not found uniform in any of the detailed studies made by this method. All computations made thus far have indicated that uniform arch loads can obtain only under the most specialized conditions. It follows, therefore, that a general expression for the proportional loads on the two systems of elements must take into account the shape of the canyon. Such an expression has not yet been developed.

Computing the deflections and proportional loads at the crown alone, or at any other one section, therefore, means little in the study as a whole. The deflections and proportionate loads must be obtained at numerous points in each system in order to obtain even an approximate solution.

If the assumption that all the load is carried by the horizontal and vertical elements is accepted, there is certainly some definite division of the load that will give equal deflections for both systems at all points. This division can be found approximately by applying a series of trial proportionate loads to each system. If enough trials be made, equal deflections for arch and cantilever elements at all points will finally be found for any assumed temperature change. The computed stresses due to these final loads, when modified by Poisson's ratio, will probably be as close to the true stresses as those computed in gravity dams. It is believed they will be more nearly correct than stresses obtained by any method that ignores the effect of the cantilever elements or computes the cantilever effect at one section only.

This method has been in development for some time and is not yet considered as entirely complete. It is believed, however, that it has reached a stage where a description of it will be found interesting. A number of dams have been analyzed and designed by this system, and the results obtained for four of them are shown in the accompanying diagrams and tables. Unfortunately, no two of them are strictly comparable because of the gradual evolution and improvement of the mathematical treatment. When the plan was first evolved it was considered impracticable to do more than roughly approximate the actual conditions. For example, in the first analysis attempted, the stresses were computed on the assumption that reinforcing steel would have to be used to resist tension in the concrete and transmit tensile stresses across construction joints. As the methods of computation were improved during further studies it was found possible to go a step further and eliminate computation of tension in the arch elements. This was done by varying the thickness of the arch and by assuming definite interior arches of regular shape at elevations where the first computations indicated tension. These interior arches are completely enclosed within the constructed arches and are entirely in compression. The largest possible interior arch that could be drawn without showing tension, was used in the final analysis. Later, it was found possible to consider the arch action to be confined entirely to the irregular shape, whatever it might be, of the uncracked concrete that forms the actual interior arch. The load is assumed as applied to the constructed arch but entirely carried by the interior one. This method is now in use and the mathematics,

as given in the Appendix, has been adopted for the analysis of these irregularly shaped arches. The part of the base of the cantilever not in compression is likewise excluded in computing stability. No tensile stresses are now assumed in either cantilever or arches. This is believed to be essential for a correct analysis as it is practically impossible to secure an adequate anchorage for reinforcing steel in the arch abutments or in the bases of the cantilevers.

TABLE 1.—ARCH DATA, DAM NO. I.

Elevation, in feet.	Thickness, in feet.	Angle.	Up-stream radius, in feet.	Down-stream radius, in feet.
4 830.0	5.00	141° 0′	197.08	192.08
4 845.4	5.00	141° 0′	197.08	192.08
4 815	7.84	108°12′	193.02	185.18
4 805	9.66	95°30′	189.15	179.49
4 795	11.48	83°00′	183.49	172.01
4 785	13.30	79°16′	177.10	163.80
4 775	15.12	77°30′	170.28	155.16
4 765	16.94	74°48′	162.80	145.86
4 755	18.75	72°08′	154.63	135.88

Fig. 2 shows the layout and profile of Dam No. I, and Table 1 contains the arch data. This is a small dam designed and built in accordance with the cylinder formula for a stress of 25 tons per sq. ft. It is the first dam to be investigated by the trial load method and the only one in which tensile stresses were considered.

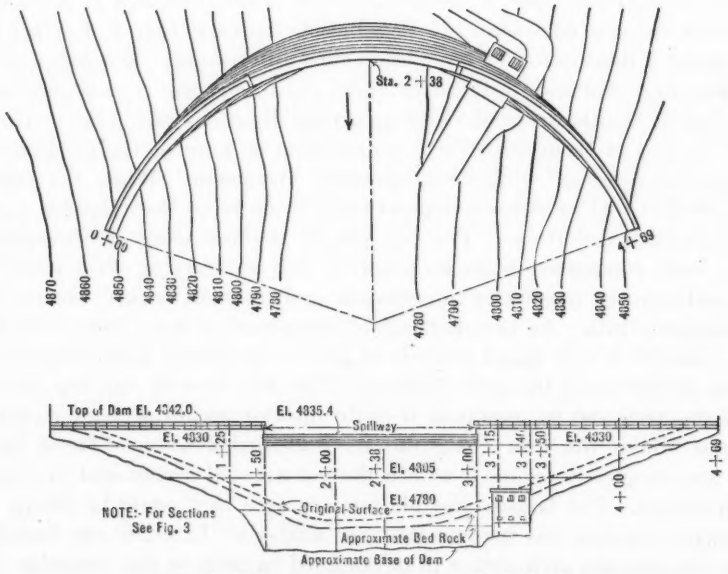

FIG. 2.—PLAN AND ELEVATION OF DAM NO. I.

Figs. 2 and 3 show this dam to be "fixed" at the base and abutments to an unusual degree. In this study the assumption was made that both arches and

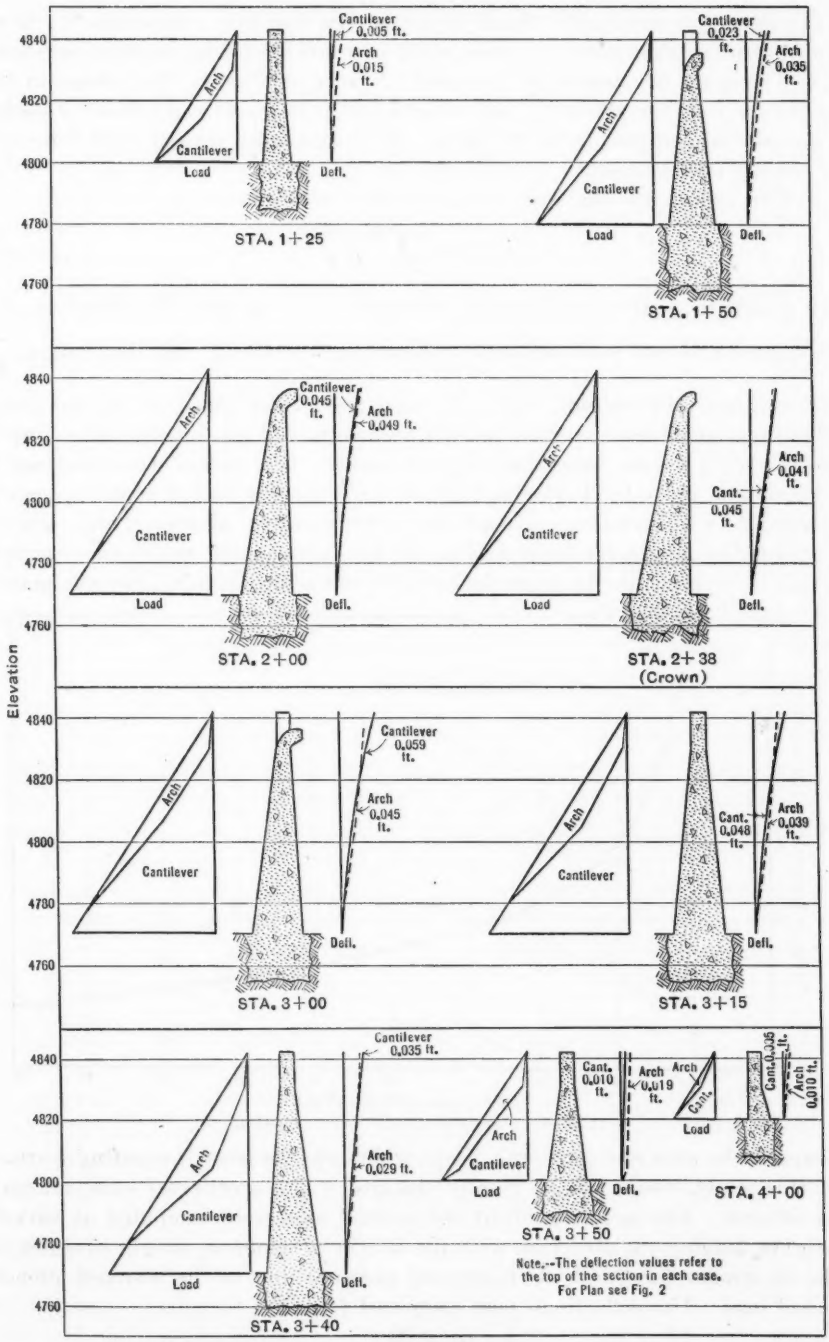

FIG. 3.—LOADS, SECTIONS, AND DEFLECTIONS OF DAM NO. I.

cantilevers were actually "fixed" at abutments and bases, respectively. It was also assumed that tensile stresses could be provided for by reinforcing steel.

The range in temperature assumed is shown on Fig. 4. Only drops in temperature were considered. The water level in this and the following designs was assumed at the top of the dam. Nine cantilever sections and three arch elements were studied.

The arch deflections were computed from the equation,

$$\Delta y = \int \frac{M x \, d s}{E I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which, M is the total bending moment at any point. The abscissa, x, is measured from the point where the deflection is desired. In this design, $\frac{d s}{E I}$ is constant. Deflections were computed from each abutment to the crown. This was made necessary by the non-symmetrical loads on the arches, which were caused by the irregularly shaped profile. The arches were analyzed by the method given by Taylor and Thompson,* thereby introducing an error in regard to rib-shortening as explained subsequently. However, this error is appreciable only in the lower arches. If the mathematics now used were to be applied to this dam the numerical results would be different, but the general

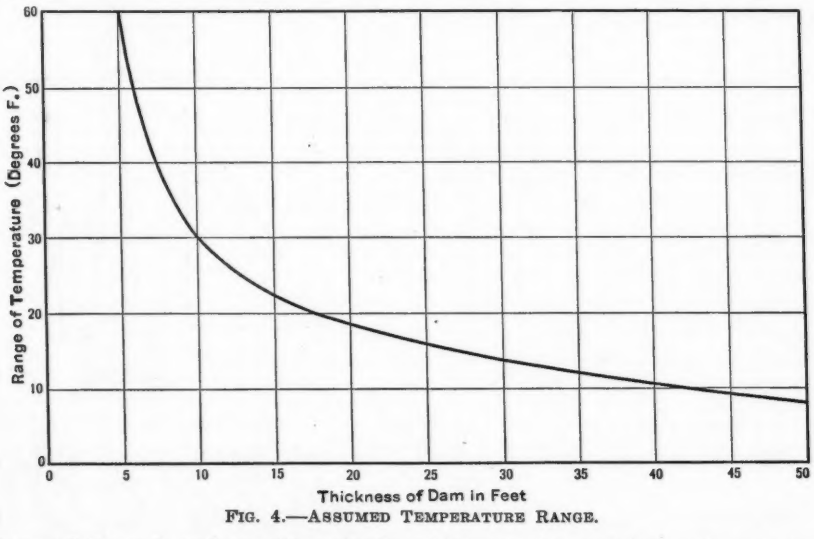

FIG. 4.—ASSUMED TEMPERATURE RANGE.

shape of the arch and cantilever loads, which are the most interesting features of this study, would not be greatly changed. The cantilevers were analyzed as follows: The moments about the neutral axis were computed at various heights, loading the structure with the weight of concrete, weight of water on the up-stream face, and the horizontal pressure due to the assumed proportional load. The deflections were computed from the formula,

$$y = \int \int \frac{M}{E I} d x \, d x$$

* "Concrete Plain and Reinforced."

Deflections due to shear are also included in studies for the latest dams. The computations for the cantilevers in this dam are somewhat more simple than in a dam without an overflow spillway notch. In the latter type, the top arches throw a negative load on the top of the cantilever because they are under little or no hydraulic head. The cantilevers, conversely, throw a load on the top arches. In this dam with water at Elevation 4842 the top arch is under a head of about 7 ft. and deflects correspondingly. No negative load, therefore, was applied to the cantilevers in this dam.

The modulus of elasticity, E, was taken as constant at 288 000 000 lb. per sq. ft. for both arches and cantilevers. There is, of course, much uncertainty as to the actual value of E. This uncertainty applies to all problems in concrete design and the usual assumption of a constant, E, was followed, therefore, in these studies.

Fig. 5 (*a*) and Fig. 5 (*b*) show two trial loads and the final load for the arches at Elevation 4 830. It is interesting to note the effect of the canyon profile on the shape of the arch loads, especially at Station 3 + 50. The abrupt change in the load line between Voussoir VI and VII is due to the sudden break in the profile between Station 3 + 40 and Station 3 + 50. The diagrams also show the extreme sensitiveness of a long thin arch to a change of loading, especially if the loading is non-uniform.

After numerous trials, four for Elevation 4 805 and twelve for Elevation 4 830, the deflection lines were brought together as shown in Figs. 3 and 5. It was clearly indicated that if the process were continued a closer agreement would result. The poorest is at Station 3 + 50. This short stiff cantilever is only 10 ft. from one of maximum height. The deflection of the two under approximately the same loads cannot be even approximately equal. The arches cannot adjust themselves well to the difference in deflection in the short distance of 10 ft. At this point there will be a complicated condition of stress. Such abrupt changes should always be eliminated if possible.

The greatest difficulty found in adjusting the trial arch loads was in the upper arches near the abutments. The long thin arches are very sensitive, and slight changes of loads gave very great changes in deflections. Relatively large up-stream deflections frequently result from slightly excessive loads at the haunches and *vice versa*.

The computed maximum stress was very high, very much higher than that given by the cylinder formula. It is believed that the constructed dam is stressed beyond the computed maximum, because it was built without reinforcement and, as tension is not taken care of, the arches have cracked. Therefore, they are resisting the compressive stresses on reduced dimensions. This condition is believed to be common to all dams designed as cylinders. On all the following designs it was found desirable to increase the arch thickness toward the abutments in order to obtain lower stresses.

Dam No. II is a large arch dam, as yet in the preliminary design stage. It is interesting because of the very uniform V-shaped profile shown in Fig. 6. The arch data are arranged in Table 2. A dam designed by the cylinder formula was first proposed for this site. It was later analyzed by the trial

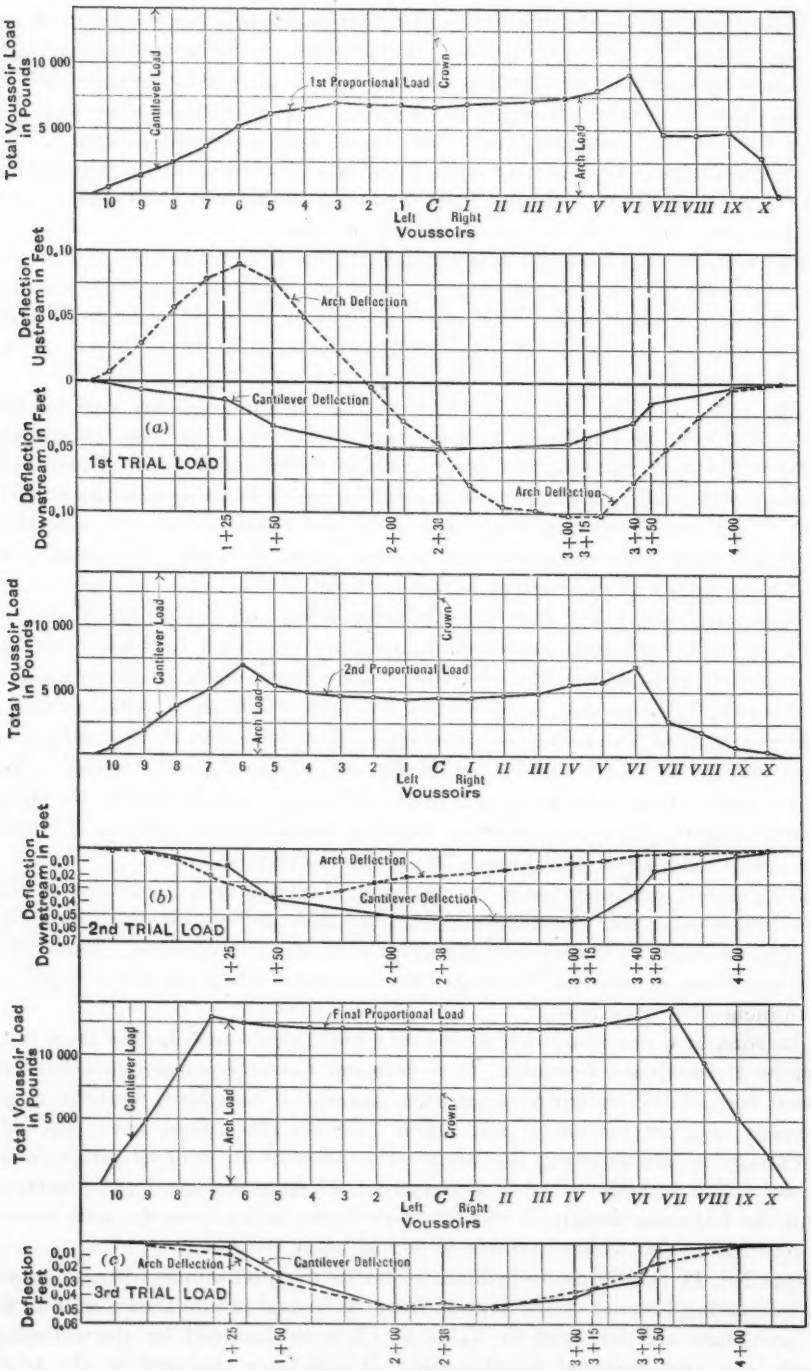

FIG. 5.—LOAD DEFLECTION DIAGRAM FOR DAM NO. I.

load method, which showed that exceedingly high stresses would occur due to the faulty distribution of the concrete.

The design was then changed to that shown in Figs. 6 and 7. It was assumed that the profile would be made actually symmetrical by excavation and concrete footings as shown in Fig. 6. This assumption reduced the work of analysis by one-half, as it was only necessary to compute the deflections of one-half the various arches and their respective cantilevers.

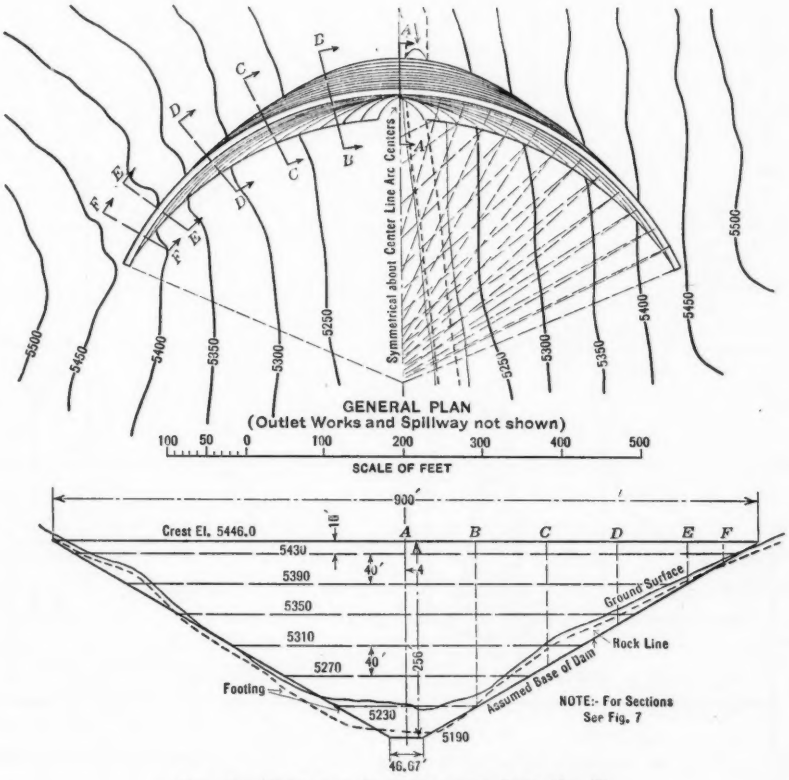

FIG. 6.—PLAN AND ELEVATION OF DAM NO. II.

The size of both systems of elements and the small depth of rock excavation precluded either system from being considered "fixed" in the sense that tension could be resisted by the grip of the rock on the concrete. Therefore, the arches were shaped so as to carry their loads entirely in compression. To accomplish this they were gradually thickened from crown to abutments, except at the top elevation. Both intrados and extrados are circular segments having different radii and centers. The cantilevers also were considered as incapable of resisting tension not only at their base but at any point. Unless reinforcing steel is used it is difficult to believe that tension could ever develop in a dam due to the opening of the construction joints, even if it were considered that the concrete itself could resist tension. To simplify the calcula-

tions the cantilevers for this and other dams were assumed to be 1 ft. thick throughout on lines radial to the crest of the dam. This is not strictly correct. The sections, except at the crown, are not exactly radial at the faces of the

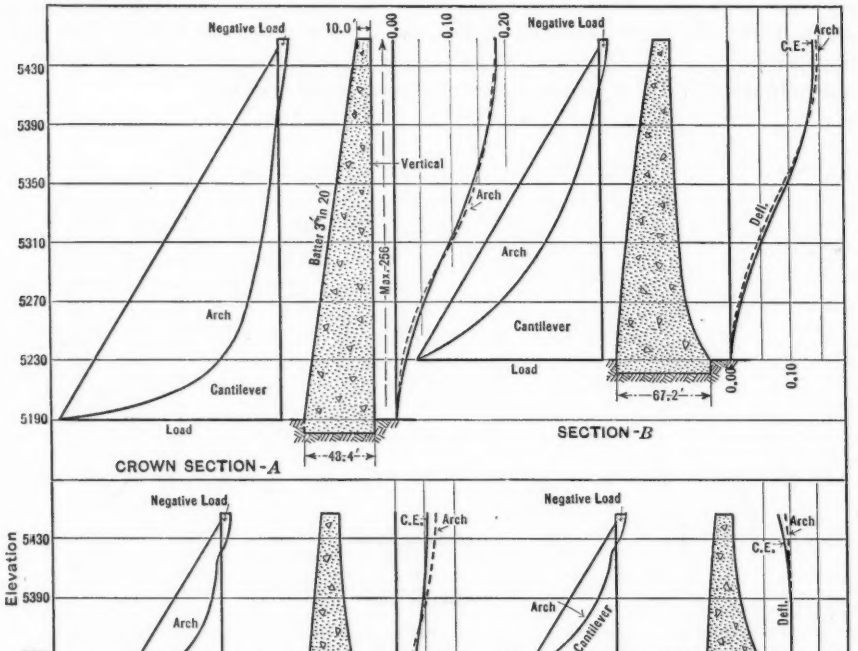

FIG. 7.—SECTIONS, LOADS, AND DEFLECTIONS, DAM NO. II.

dam and the cantilevers are not of uniform thickness. The designs made thus far include whatever errors are due to the inaccurate assumptions mentioned.

The cantilever moments included the weight of concrete, water on the up-stream face, the negative load due to the unloaded arches at the top, and the assumed proportional part of the water load. These forces were taken as acting about the neutral axis of the sections at the various elevations considered. In cases where the resultant fell out of the middle third of the section, the moment of inertia was taken only for the portion of the section in compression, and the moments were considered as acting about the neutral axis of the compressed portion. The deflections were computed for each 20 ft. in height.

TABLE 2.—ARCH DATA, DAM NO. II.

Elevation, in feet.	Thickness of crown, in feet.	Angle.	Up-stream radius, in feet.	Down-stream radius, in feet.
5 446	10.0	132° 54′	388.00	378.00
5 430	12.4	126° 04′	386.45	369.28
5 410	15.4	117° 32′	384.11	357.39
5 390	18.4	109° 00′	381.20	343.55
5 370	21.4	100° 27′	377.37	326.61
5 350	24.4	91° 54′	372.84	308.91
5 330	27.4	83° 27′	366.84	285.15
5 310	30.4	75° 00′	358.14	256.70
5 290	33.4	65° 56′	351.69	228.00
5 270	36.4	56° 52′	337.17	196.70
5 250	39.4	48° 40′	316.99	143.11
5 230	42.4	40° 28′	284.73	92.77
5 210	45.4	32° 00′	234.65	53.33
5 190	48.4	.....	.....	.....

As in the study for Dam No. I it was found much easier to adjust the loads for the lower elevations than for the higher. The top arches being only about 10 ft. thick and about 900 ft. long were very sensitive. The high cantilevers having the resultants outside the middle third were also very unstable. Moreover, the long lever arm of the negative load on the cantilevers (produced by the unloaded top arch) caused large changes in deflections with very small changes in the negative loads.

In the three lower arches (Elevations 5 310, 5 270, and 5 230) the thickness required to avoid an overhang at the higher portions was too great to avoid tension entirely. This tension develops on the extrados at the abutments and on the intrados at the crown. Interior arches, therefore, were assumed to form within the constructed arch and to act entirely in compression. The deflection of these interior arches should coincide with that of the cantilevers. It was in the analysis of the thick lower arches that the error in the usual method of computing rib-shortening was discovered. This error consists in computing the rib-shortening thrust from the average thrusts due to loads and temperature, without including the rib-shortening itself. If the rib-shortening is not included in obtaining the average, there will be negative thrusts in short thick arches. This error was corrected by a modification of the rib-shortening formula and the remaining arch analyses were similar to those used for Dam No. I. The computed results are believed to be quite close to those which would be obtained by the present mathematical treatment.

FIG. 8.—LOAD AND DEFLECTION CURVES, DAM NO. II.

Fig. 7 shows the vertical sections, the final proportionate water loads carried by each system, the negative load at the top, and the computed deflections.

Fig. 8 (*a*) to Fig. 8 (*g*) show the horizontal shape of the final arch loads and the deflections of the arches and cantilevers. Fig. 9 shows the developed elevation of the dam on which are drawn lines of equal deflections as computed for the arches. Fig. 10 shows the assumed interior arches for the three lowest elevations. They were found by removing from the original arches the parts in tension, and drawing symmetrical arches in the unarched portion. The reduced arches were then analyzed. This method is only approximate. It does not give definitely the largest interior arch that is free from tension.

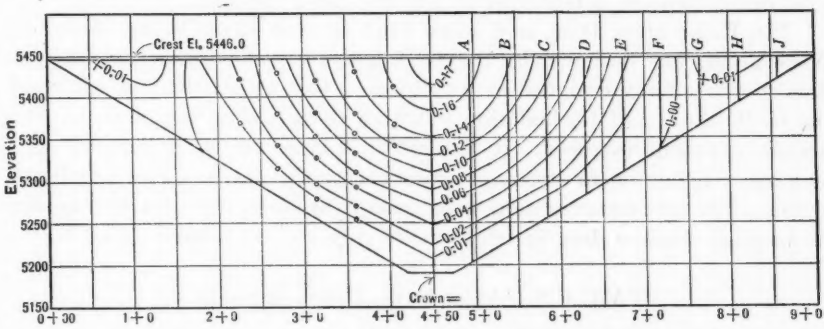

FIG. 9.—DEVELOPED ELEVATION OF DAM NO. II, SHOWING LINES OF EQUAL DEFLECTION (DEFLECTIONS, IN FEET).

At the time the studies of this dam were being made, it was considered too laborious to analyze the irregularly shaped interior arches. In the succeeding designs, however, this was done.

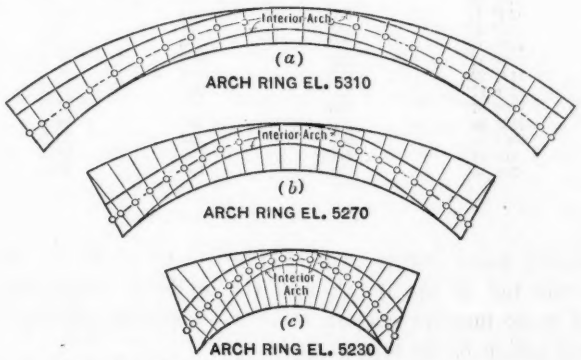

FIG. 10.—ASSUMED INTERIOR ARCHES, DAM NO. II.

The formation of the interior arches has been well stated by L. J. Mensch,* M. Am. Soc. C. E., William Cain,† M. Am. Soc. C. E., and also by B. F. Jakobsen,‡ M. Am. Soc. C. E.

* *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 308.
† *Loc. cit.*, p. 310.
‡ *Loc. cit.*, Vol. 90 (June, 1927), p. 475.

The greatest discrepancy between the cantilever and arch deflections is shown in Fig. 8 (*a*) and Fig. 8 (*b*) for Elevations 5 446 and 5 430. This is also shown in Fig. 7 for Cantilevers *D*, *E*, and *F*. These lines could have been brought more closely together by further trials. However, those already made, ten or more for each arch, showed relatively great variation in deflection and small changes in stress. Therefore, the further effort was not considered worth while.

As stated before, this design is a preliminary one only. It was made as an attempt to improve the original cylinder design. The saving in concrete amounts to 25%, and the stresses are less than one-half those in the cylinder designs as determined by the trial load method.

The Horse Mesa Dam, now being built by Salt River Valley Water Users Association, was designed by the trial load method. (See Table 3.) This dam was originally designed as a cylinder. The design was not approved by the U. S. Bureau of Reclamation to which it was submitted, and the present design was then developed. It resulted in a large saving in concrete as well as a great reduction in stress. It assumes no tension in either cantilever or arches. The mathematics used were practically those given in the Appendix. As in other dams, a drop in temperature only was considered. (Fig. 4.)

TABLE 3.—ARCH DATA, HORSE MESA DAM.

Elevation of crest.	Central angle.	Up-stream radius, in feet.	Down-stream radius, in feet.	Arch Thickness, in Feet.	
				Crown.	Springing line.
1 921	112° 30′	240	232	8.0	8.0
1 897	114° 47′	235	218	11.6	14.4
1 875	116° 45′	231	206	14.8	20.1
1 850	119° 18′	227	193	18.6	26.8
1 825	121° 11′	224	181	22.3	33.4
1 800	118° 26′	220	168	26.0	40.9
1 775	117° 17′	216	157	29.0	46.0
1 750	116° 57′	213	146	31.8	51.5
1 725	113° 32′	210	136	34.8	56.5
1 700	106° 11′	207	123	36.5	61.0
1 675	91° 31′	202	113	38.5	59.8
1 650	76° 45′	195	102	40.2	57.3

The maximum water surface was assumed to be at the top of the parapet (5 ft. above the top of the dam), and the spillway structures at the ends were assumed to be massive enough to offer resistance sufficient for the dam to develop arch action to the top.

This design proved to be well proportioned, the average compression in the arches being 286 lb. per sq. in., and in the cantilevers, 222 lb. per sq. in. The difference in the maximum compression of the two elements was only 12 lb. The profile of this dam is irregular as was the case with Dam No. I, and again this irregularity is strikingly reflected by the shape of the arch loads.

On account of the thin section at the crown practically no load is taken by the cantilever. This condition exists at the abutments where, on account

of the almost perpendicular sides, the cantilevers cannot deflect; between these points the cantilevers carry a fair proportion at the bottom, but practically nothing at the top, of the dam.

In the analysis of this dam the effective section of the arches was considered to be only that portion in compression. Where tension developed, it was eliminated by assuming reduced sections until one was found that had practically a zero stress along the lines of the discarded areas. This condition occurred only in the lower portion of the dam where the arches are relatively thick. These reduced sections are shown in Fig. 11 and are not symmetrical due to the unsymmetrical loading of the arches.

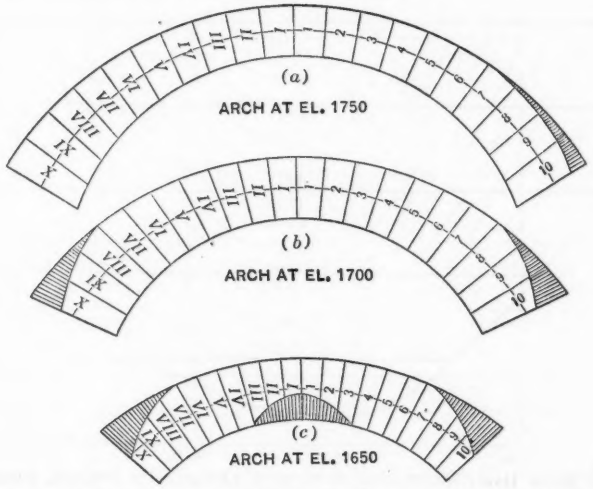

FIG. 11.—PLANS OF ARCHES AS ASSUMED TO ELIMINATE TENSION, HORSE MESA DAM.

Analysis discloses that, in this case, the maximum deflection does not occur at the crest, but at a point about 45 ft. below the crest and near the crown of the arch. (See Fig. 12.) This was to be expected as the only load on the arch at the top is that due to the negative load on the cantilevers. A similar condition of deflection was deduced by Mr. Alfred Stuckey in his analysis of the Jogne Dam, Switzerland.* Since the designs described were made, the dimensions have been changed slightly to conform with conditions that have developed in the process of construction.

Table 4 gives the results of the analysis of the Gibson Dam on Sun River, Montana, which is now being built by the U. S. Bureau of Reclamation. This site has been under consideration for a long time, but hitherto only for a gravity dam. The shape of the profile does not clearly indicate the suitability of an arch dam. However, by taking advantage of the effect of the cantilever action in reducing water loads, as determined by the trial load method, the present design is responsible for a saving of more than 41 000 cu. yd. of concrete over the gravity design. In contrast to the three foregoing dams, the Gibson Creek Dam has a vertical up-stream face and a constant up-stream

* "Study of Arched Dams," *Bulletin technique de la Suisse Romande*, 1922.

radius. The previously considered designs are of the variable radius type. Because of the wide flat-bottomed profile (Fig. 13), a large proportion of the cantilevers are of approximately maximum height. In a variable radius design the crown cantilever has a vertical down-stream face and an inclined up-stream one. The adjacent sections approximate this shape. They are very inefficient in resisting thrusts on the up-stream face. The same area

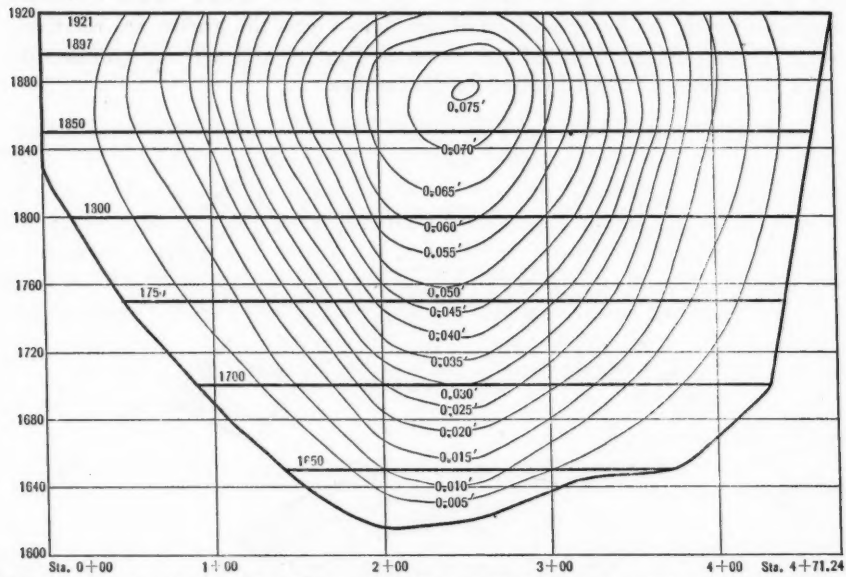

FIG. 12.—HORSE MESA DAM: DEVELOPED ELEVATION LOOKING UP STREAM, LINES OF EQUAL DEFLECTION.

with vertical up-stream and inclined down-stream faces is much stiffer. Therefore, if cantilever action is to be considered in profiles similar to that of the Gibson Dam, it will usually be found that the vertical up-stream face, or constant-angle type, is the most economical. This is not true, however, with narrow V-shaped profiles where the bottom arches are short and relatively more efficient, and the maximum height obtains for only a small proportion of the profile.

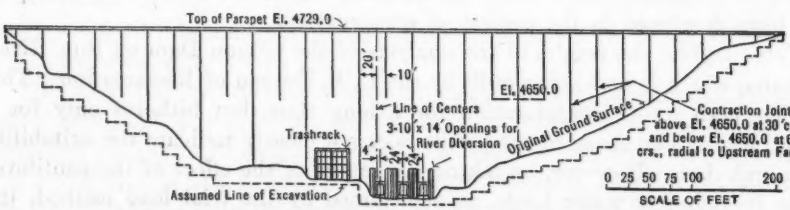

FIG. 13.—DEVELOPED UP-STREAM ELEVATION, GIBSON DAM.

The mathematics used are those given in the Appendix and represent the latest development. The maximum computed stress is 600 lb. per sq. in. and occurs at Elevation 4 550. The maximum arch stress is 300 lb. per sq. in.

and occurs at the crown at Elevations 4 675 and 4 650. Several arches at the lower elevation were analyzed with their full water load. These computations showed a maximum stress of approximately 1 000 lb. per sq. in. The computed stresses on this dam are believed to be the most accurately determined of all those studied thus far.

TABLE 4.—ARCH DATA, GIBSON DAM.

Station on line of centers.	Elevation, in feet.	Thickness on line of centers, in feet.	Down-stream radius, in feet.
0 + 00*	.....	.....	.....
0 + 00	4 725.5	15.0	390.00
0 + 00	4 717.5	16.57	388.43
0 + 07.5	4 700.0	20.0	377.5
0 + 18	4 675.0	25.0	362.0
0 + 34	4 650.0	30.0	341.0
0 + 48.8	4 625.0	36.2	320.0
0 + 66†	.....	.....	.....
0 + 74.5	4 600.0	44.5	286.0
0 + 96.9	4 575.0	55.1	253.0
1 + 17.0	4 550.0	71.5	216.5
1 + 17.0	4 530.0	87.0	201.0

* Center for up-stream radius of 405 ft.
† Station 5 + 34 base line. Central angle equals 24° 00′

In all the dams described, provision was made for leaving openings for closing plugs. These plugs are poured in the coldest weather practicable, and as long a time as possible after the concrete in the dam has been placed so as to compensate, to some degree, for the shrinkage of the concrete. Other than this no consideration has been given to shrinkage.

In this design, due to the large relative importance of the cantilever action as compared to the arches, an arbitrary assumption for uplift was made. It was considered that the full head at the up-stream face diminished to zero or tail-water elevation at the down-stream face and acted over one-half the base of the section. In addition, a drainage system was provided.

No attempt was made in any of these studies to consider foundation deformation or swelling of the concrete due to water soakage. Both these actions undoubtedly modify deflections and stresses to some degree. The writers believe that an attempt to include foundation deformation would be unwarranted from a practical standpoint because of the indeterminate and, in many cases, the variable bearing power of the rock itself.

At present, so little is actually known about the changes in volume of concrete due to water soakage that the writers have not tried to include its effect in this method. If, however, one should desire to provide for the effect of water soakage there is no reason why the additional assumption could not be incorporated in the trial load method.

The action of gravity, however, is definitely known. The resistance offered by the cantilevers can be determined probably more accurately than that of the arches. The writers believe that action of cantilevers should not be neglected in designing arch dams. They certainly carry a part of the load and just as certainly modify the arch loads; and this fact should be recognized

in designing the arches. The trial-load method offers a practical method of doing this.

Conclusions

In conclusion, the writers desire to repeat that this paper is not offered as a rigid and complete solution of the problem of arch dam design, but as a step toward it.

It is believed, however, that four conclusions are indicated:

(1) If the cantilevers act the arch elements are not under uniform loads.

(2) Computing proportional loads and deflections for each system at the crown, or at any other one point, means little or nothing in the solution of the problem as a whole.

(3) Rational formulas for a complete solution must include terms that express the shape of the canyon.

(4) An approximately accurate analysis can be made by the use of the method herein proposed.

The designs and studies of all the dams described were made under the direction of J. L. Savage, M. Am. Soc. C. E., Designing Engineer, U. S. Bureau of Reclamation. Julian Hinds, M. Am. Soc. C. E., Assistant Designing Engineer, developed the final form of the mathematics and assisted throughout in perfecting this method. I. E. Houk, M. Am. Soc. C. E., J. J. Doland, Assoc. M. Am. Soc. C. E., and Mr. R. E. Glover, Engineers, U. S. Bureau of Reclamation, assisted in the designs of the Gibson Dam.

APPENDIX

The following discussion involves the application of the elastic theory to arches both symmetrical and unsymmetrical in form. The underlying principles are the same as those given in various textbooks on concrete. The formulas deduced in textbooks, however, are generally subjected to simplifications that limit their usefulness in the study of arched dams. An attempt is here made to develop the formulas in a perfectly general way. The resulting equations are rather forbidding in appearance, but are, in fact, comparatively simple to apply.

Fig. 14 represents one side of an arch, the influence of the other side upon it being replaced by the introduction of proper forces or reactions at the crown. A load, W, placed on the arch will cause the crown point of the arch axis to be displaced. The displacement will be due partly to the bending of the arch ring, and partly to a change in the length of the center line of the arch.

In general, a load placed on the arch will produce a moment at every point along the ring. These moments cause the arch ring to bend.

The arch may be considered as made up of a series of short sections, or voussoirs. The bending of any short length, such as that having its center at D, will cause an angular displacement of the part ring, $A\ C$, and a displace-

ment, in space, of the crown point at C. The movement at C will be equal to $E\,C\,\Delta\,\theta$, plus the actual displacement of the point, E, $\Delta\,\theta$ being the change in angle between the two end faces of the voussoir. If the voussoirs are short, the movement is approximately $D\,C\,\Delta\,\theta$. From mechanics:

$$\Delta\,\theta = \frac{M\,s}{E\,I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which, M is the bending moment, s is the length of the voussoir, E is the modulus of elasticity, and I is the moment of inertia of the arch ring. Equation (1) is strictly accurate only when M and I are constant throughout the voussoir, or when the length, S, is infinitesimal. However, it is assumed to serve with sufficient accuracy in practical cases if average values of M and I are taken for each voussoir. A further simplification is often made by assuming that M and I computed at the middle of the voussoir are equivalent to the average values for the voussoir.

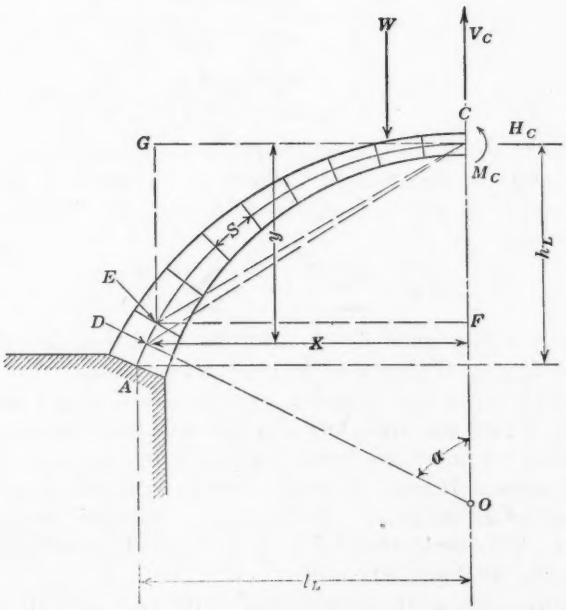

FIG. 14.

The angular movement, $\Delta\,\theta$, will be considered positive in a contra-clockwise direction, and M will be positive when the resulting tension is on the intrados side of the arch ring.

The path of motion at C will be determined by the direction of the line, $D\,C$. The use of angles is avoided by considering the vertical and horizontal motions separately. The vertical motion of the point, C, will be approximately equal to that of the point, F, or

$$\Delta\,y = x\,\Delta\,\theta = x\,\frac{M\,s}{E\,I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which, Δy is the vertical movement of C, considered positive when upward; and x is the horizontal distance from C to D, considered positive regardless of direction of measurement.

The horizontal movement of C will be approximately equal to that of the point, G, or

$$\Delta x = -y \Delta \theta = -y \frac{M s}{E I} \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

in which, Δx is the horizontal movement of C, considered positive to the right, and y is the vertical distance from D to C, considered positive in all cases. The terms, horizontal and vertical, are used for convenience, although in arched dams the ordinate, y, is never vertical.

If Equations (2) and (3) are applied successively to each voussoir from A to C and the results added algebraically, the total movement of the point, C, due to bending, will be determined. The process may be expressed as follows:

$$\Delta y = \sum_C^A x \frac{M s}{E I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

$$\Delta x = -\sum_C^A y \frac{M s}{E I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

In addition to the horizontal and vertical movements there is an angular motion, or change in direction of tangent at C, which is the sum of the angular movements in all the voussoirs from A to C. This movement may be expressed thus:

$$\Delta \phi = \sum_C^A \Delta \theta = \sum_C^A \frac{M s}{E I} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

There is also a movement due to shearing distortion, but this is usually neglected as being small compared to other movements.

The moments, M, in the foregoing equations involve not only the known external loads, W, on the arch, but also the unknown reaction at C. This reaction consists of a single force, unknown in amount, direction, and point of application. It may be assumed to be replaced by a horizontal or axial force applied at the center of the ring, a transverse or shearing force, and a moment. This replacement does not reduce the number of unknowns, but simplifies the arithmetical work.

If x and y are taken as the co-ordinates of the arch axis, then the moment of all forces about any given point between A and C (Fig. 14) will be:

$$M = M_C + H_C y + V_C x - M_L \quad \ldots\ldots\ldots\ldots\ldots\ldots (7)$$

in which, M_C, H_C, and V_C are, respectively, the moment, thrust, and shear at the crown, and M_L is the moment, about the point in question, of all external loads, W, between the crown and the point. All factors are considered positive when acting as shown in Fig. 14. The use of the minus sign before M_L in this case is not strictly mathematical, but conforms to usual practice Actually, M_L is added algebraically, but since it is in all usual cases a negative quantity, it is customarily computed without regard to algebraic sign and then subtracted.

Substituting this value for M in Equations (4), (5), and (6), yields three equations, as follows:

Change in y at crown,

$$\Delta y = \sum_C^A \frac{x\,s}{E\,I}(M_C + H_C\,y + V_C\,x - M_L) \ldots\ldots\ldots\ldots (8)$$

Change in x at crown,

$$\Delta x = -\sum_C^A \frac{y\,s}{E\,I}(M_C + H_C\,y + V_C\,x - M_L) \ldots\ldots\ldots\ldots (9)$$

Change in direction at crown,

$$\Delta \phi = \sum_C^A \frac{s}{E\,I}(M_C + H_C\,y + V_C\,x - M_L) \ldots\ldots\ldots\ldots (10)$$

Equations (8), (9), and (10) give the movement at C produced by bending only, the length of the center line being assumed to be constant. If this length changes, as it always does in practice, there will be an additional movement. If the length, $A\,E$, is changed by an amount, $\Delta\,S$, other things remaining constant, the point, E, and, consequently, the point, C, will move vertically an amount:

$$\Delta y = \Delta s \sin \alpha \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (11)$$

in which, α is the angle that a radius to the point, D, makes with the crown radius.

Likewise, the resulting horizontal movement will be,

$$\Delta x = \Delta s \cos \alpha \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (12)$$

The total resulting movement at C will be the summation of the movements due to the lengthening or shortening of all the voussoirs between A and C, or,

$$\Delta y = \sum_C^A \Delta s \sin \alpha \ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

$$\Delta x = \sum_C^A \Delta s \cos \alpha \ldots\ldots\ldots\ldots\ldots\ldots\ldots (14)$$

A change in the length of the voussoirs may be caused by a change in temperature or by the compression (or extension) of the fibers by an axial thrust. The movement caused by a temperature change is readily determined. The total vertical movement at the crown is,

$$\Delta y = h_L\,c\,t \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (15)$$

in which, h_L is the rise of the arch, c is the coefficient of temperature expansion, and t is the temperature change.

Likewise, if l_L is the length of the half span, the horizontal movement at the crown is,

$$\Delta x = l_L\,c\,t \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

The change in length in any voussoir due to thrust is:

$$\Delta s = -\frac{H s}{E\,T} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

in which, H is the average axial thrust throughout the voussoir, and T is the average radial thickness of the arch ring, the thickness normal to the

plane of the paper in Fig. 14 being unity. The minus sign is required if a compressive thrust and a lengthening of the fibers both are considered positive. The value of H at any point is the combination of the components of H_C, V_C, and the external loads between the crown and the point, acting in the direction of the axis at the point. If H_L be chosen to represent the sum of the components of all external forces between the point and the crown, parallel to the axis at a given point, the thrust at that point is:

$$H = H_C \cos \alpha - V_C \sin \alpha + H_L \ldots\ldots\ldots\ldots\ldots\ldots (18)$$

Equation (17), therefore, may be written thus:

$$\Delta s = -\frac{s}{E\,T}(H_C \cos \alpha - V_C \sin \alpha + H_L)\ldots\ldots\ldots\ldots (19)$$

Inserting this value for ΔS in Equations (13) and (14), the total movement of C due to elastic change in length of center line of arch may be expressed thus:

$$\Delta y = -\sum_C^A \frac{s \sin \alpha}{E\,T}(H_C \cos \alpha - V_C \sin \alpha + H_L)\ldots\ldots (20)$$

$$\Delta x = -\sum_C^A \frac{s \cos \alpha}{E\,T}(H_C \cos \alpha - V_C \sin \alpha + H_L)\ldots\ldots (21)$$

Adding Equations (8), (15), and (20), the total vertical movement at C is found to be:

$$\Delta y = \sum_C^A \frac{x\,s}{E\,I}(M_C\,H_C\,y + V_C\,x - M_L) + h_L\,c\,t$$
$$- \sum_C^A \frac{s \sin \alpha}{E\,T}(H_C \cos \alpha - V_C \sin \alpha + H_L)\ldots\ldots\ldots (22)$$

The total horizontal movement, found by adding Equations (9), (16), and (21), is:

$$\Delta x = -\sum_C^A \frac{y\,s}{E\,I}(M_C\,H_C\,y + V_C\,x - M_L) + l_L\,c\,t$$
$$- \sum_C^A \frac{s \cos \alpha}{E\,T}(H_C \cos \alpha - V_C \sin \alpha + H_L)\ldots\ldots\ldots (23)$$

Equations (10), (22), and (23) are independent, but cannot be solved since they contain the six unknowns, Δx, Δy, $\Delta \phi$, H_C, V_C, and M_C. However, the right-hand half of the arch may be treated in the same manner and three similar equations obtained involving the same six unknowns, thereby making it posssible to determine the reaction at the crown.

The equations for the right-hand part of the arch will be identical with those just developed for the left-hand part, except as to certain signs. A positive moment on the right will cause rotation in a clockwise, or negative, direction. For this reason, $\Delta \theta$ in Equation (1) and $\Delta \phi$ in Equations (6) and (10) become negative. Since distances and co-ordinates are considered positive regardless of direction, the sign of Δy will not change in any equation. A positive moment will produce a Δx-movement toward the right. Therefore, the negative sign will be dropped in Equations (3), (5), and (9). Since a positive shear is assumed to re-act upward on the left-hand part of the

arch, it must re-act downward on the right-hand half. It will, therefore, produce a negative bending moment, and the term, $V_C\,x$, in Equation (7) will become negative. Likewise, the effect on the thrust will be reversed and the V_C sin α-term in Equation (18) will become positive. The effect of $\Delta\,s$ on $\Delta\,x$ will be reversed, and a reversal of sign is required in Equations (12), (14), and (16). When these changes have been made and carried through the derived equations, a new set of equations applicable to the right side of the arch are obtained.

Equations (10), (22), and (23), on the left side, and the similar equations, (10), (22), and (23), on the right side, are independent, and contain only six unknowns. They are, therefore, capable of simultaneous solution. The crown movements, $\Delta\,y$, $\Delta\,x$, and $\Delta\,\phi$, are necessarily equal for the two sides of the arch. They may be eliminated at once by equating Equations (10) left, and (10) right, (22) left, and (22) right, and (23) left, and (23) right. The resulting equations are as follows:

$$\sum_C^A \frac{s}{E\,I}(M_C + H_C\,y + V_C\,x - M_L)$$
$$= -\sum_C^B \frac{s}{E\,I}(M_C + H_C\,y - V_C\,x - M_R) \quad \ldots\ldots\ldots\ldots (24)$$

$$\sum_C^A \frac{x\,s}{E\,I}(M_C + H_C\,y + V_C\,x - M_L) + h_L\,c\,t$$
$$-\sum_C^A \frac{s \sin\alpha}{E\,T}(H_C \cos\alpha - V_C \sin\alpha + H_L)$$
$$= \sum_C^B \frac{x\,s}{E\,I}(M_C + H_C\,y - V_C\,x - M_R) + h_R\,c\,t$$
$$-\sum_C^B \frac{s \sin\alpha}{E\,T}(H_C \cos\alpha + V_C \sin\alpha + H_R) \quad \ldots\ldots\ldots\ldots (25)$$

$$-\sum_C^A \frac{y\,s}{E\,I}\Big(M_C + H_C\,y + V_C\,x - M_L\Big) + l_L\,c\,t$$
$$-\sum_C^A \frac{s \cos\alpha}{E\,T}\Big(H_C \cos\alpha - V_C \sin\alpha + H_L\Big)$$
$$= \sum_C^B \frac{y\,s}{E\,I}\Big(M_C + H_C\,y - V_C\,x - M_R\Big) - l_R\,c\,t$$
$$+\sum_C^B \frac{s \cos\alpha}{E\,T}\Big(H_C \cos\alpha + V_C \sin\alpha + H_R\Big) \quad \ldots\ldots\ldots\ldots (26)$$

If $\sum_C^A$ and $\sum_C^B$ are designated, respectively, by $\sum_L$ and $\sum_R$, Equations (23), (25), and (26) simplify as follows:

$$M_C\left(\sum_L \frac{s}{I} + \sum_R \frac{s}{I}\right) + H_C\left(\sum_L \frac{s\,y}{I} + \sum_R \frac{s\,y}{I}\right)$$
$$+ V_C\left(\sum_L \frac{s\,x}{I} - \sum_R \frac{s\,x}{I}\right) = \sum_L \frac{M_L\,s}{I} + \sum_R \frac{M_R\,s}{I} \quad \ldots\ldots (27)$$

$$M_C\left(\sum_L \frac{x\,s}{I}-\sum_R \frac{x\,s}{I}\right)+H_C\left(\sum_L \frac{x\,y\,s}{I}-\sum_L \frac{s\sin\alpha\cos\alpha}{T}\right.$$
$$\left.-\sum_R \frac{x\,y\,s}{I}+\sum_R \frac{s\sin\alpha\cos\alpha}{T}\right)+V_C\left(\sum_L \frac{x^2\,s}{I}+\sum_R \frac{x^2\,s}{I}\right.$$
$$\left.+\sum_L \frac{s\sin^2\alpha}{T}+\sum_R \frac{s\sin^2\alpha}{T}\right)=\sum_L \frac{M_L\,x\,s}{I}-\sum_R \frac{M_R\,x\,s}{I}$$
$$+\sum_L \frac{H_L\,s\sin\alpha}{T}-\sum_R \frac{H_R\,s\sin\alpha}{T}-c\,t\,E\,(h_L-h_R)\ldots\ldots(28)$$

$$M_C\left(\sum_L \frac{y\,s}{I}+\sum_R \frac{y\,s}{I}\right)+H_C\left(\sum_L \frac{y^2\,s}{I}+\sum_R \frac{y^2\,s}{I}+\sum_L \frac{s\cos^2\alpha}{T}\right.$$
$$\left.+\sum_R \frac{s\cos^2\alpha}{T}\right)+V_C\left(\sum_L \frac{x\,y\,s}{I}-\sum_R \frac{x\,y\,s}{I}-\sum_L \frac{s\sin\alpha\cos\alpha}{T}\right.$$
$$\left.+\sum_R \frac{s\sin\alpha\cos\alpha}{T}\right)=\sum_L \frac{M_L\,y\,s}{I}+\sum_R \frac{M_R\,y\,s}{I}-\sum_L \frac{H_L\,s\cos\alpha}{T}$$
$$-\sum_R \frac{H_R\,s\cos\alpha}{T}+\frac{c\,t\,E}{s}\,(l_L+l_R)\ldots\ldots\ldots\ldots\ldots(29)$$

After the constant terms and the constant coefficients of M_C, H_C, and V_C are found, Equations (27), (28), and (29) are readily solved. If desired, the solution may be made in the general form, and expressions obtained from which the unknowns can be calculated directly, but the work is not expedited by this procedure.

After M_C, H_C, and V_C are found, the moments, thrusts, and shears at other points may be computed from Equations (7) left, (7) right, (18) left and (18) right.

The horizontal and vertical components of the crown deflection are given by Equations (22) left, and (23) left, or by Equations (22) right, and (23) right. These formulas can be used for computing the deflection at points other than the crown, by carrying the summations from the abutment up only to the point at which the deflection is desired. The modulus of elasticity, E, is assumed as a constant, and may be placed outside the summation sign wherever convenient.

The deflections at intermediate points can be determined with less labor, however, by computing the movement within each voussoir separately, and taking the summation from the abutment toward the crown. The angular movement of any division, such as the one having its center at F, Fig. 14, is found by the summation of Equation (1), from A to F, thus:

$$\Delta\,\theta_F=\sum_F^A \frac{M\,s}{E\,I}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(30)$$

The actual linear movement of the upper end of the voussoir, due to this angular movement, is:

$$\Delta\,d_F=s\sum_F^A \frac{M\,s}{E\,I}\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots(31)$$

The movement, $\Delta\,d_F$, in Equation (31) is, practically, at right angles to the chord of the voussoir; that is, parallel to the radius through the center

of the voussoir. These movements for the various voussoirs are not in one direction, and cannot be added algebraically, as they stand. They may be combined graphically, or resolved into horizontal and vertical components, and then added. The components are obtained by multiplying Equation (31) by the cosine and sine of the angle to the center of the voussoir. Performing these multiplications and taking the summation from the abutment up, the total component movements of the upper end of the voussoir, F, are found to be:

$$\Delta x_F = \sum_F^A \left(s \cos \alpha \sum_F^A \frac{M s}{E I} \right) \ldots\ldots\ldots\ldots\ldots (32)$$

$$\Delta y_F = \sum_F^A \left(s \sin \alpha \sum_F^A \frac{M s}{E I} \right) \ldots\ldots\ldots\ldots\ldots (33)$$

Equations (30) to (33), inclusive, refer to movements due to bending only. The movements caused by change in length of the center line, due to thrust and temperature change are parallel to the various chords. They are, therefore, normal to the movements due to bending and must be resolved separately. After the individual components for both bending and shortening have been found, they may be added separately, and then combined, or combined and then added. Where the work is done on a computing machine it is advantageous to determine the summation separately. The total movement in any voussoir from temperature and thrust, is,

$$\Delta s = s\, c\, t - \frac{H s}{E T} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (34)$$

Taking components and summating, the total horizontal and vertical movements at the upper end of the voussoir, F, due to temperature and rib-shortening, are

$$\Delta x_F = \sum_F^A \cos \alpha \left(s\, c\, t - \frac{H s}{E T} \right) \ldots\ldots\ldots\ldots\ldots (35)$$

$$\Delta y_F = \sum_F^A \sin \alpha \left(s\, c\, t - \frac{H s}{E T} \right) \ldots\ldots\ldots\ldots\ldots (36)$$

Equations (32) and (35) and Equations (33) and (36) may be combined if desired.

All the formulas deduced have been left in the general form. If the factor, s, is made constant for the entire arch, or for each side, it may be removed from the summation, or both s and I may be so removed if $\frac{s}{I}$ is constant. The modulus of elasticity, E, is presumably a constant, and may be placed outside the summation sign wherever convenient.

These equations do not differ essentially from those ordinarily given for the elastic arch, except in regard to the allowance for the elastic shortening of the arch rib. In the analysis of concrete bridge arches it is customary to base the rib-shortening stresses on the average thrust exclusive of the rib-shortening thrust. In long flexible arches this plan is sufficiently accurate for all practicable purposes. With short stiff arches, such as those encountered at the bottom of an arched dam, the magnitude of the rib-shortening thrust

becomes appreciable, and failure to make proper allowance in computing the average thrust is likely to lead to serious error. This approximation is eliminated from the equations herein deduced.

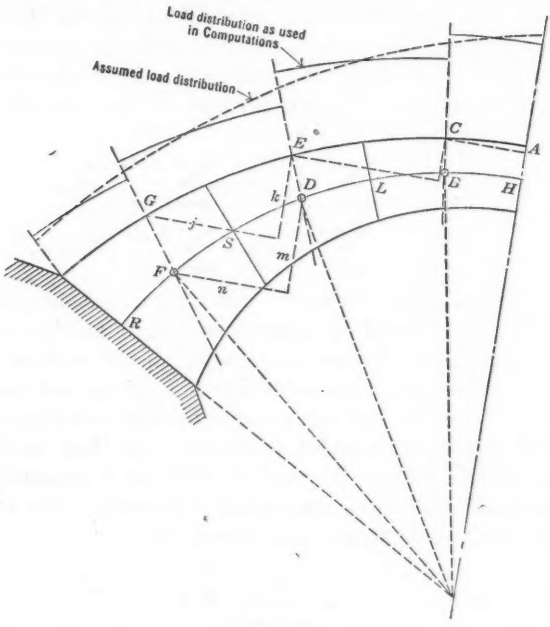

FIG. 15.

Applying this rule to Fig. 15, and considering the horizontal and vertical forces separately, the moment at F is, evidently,

$$M_F = M_D + n \sum_D^H p\ X + m \sum_D^H p\ Y + \frac{1}{2}\, n\, p\, j + \frac{1}{2}\, n\, p\, k \ldots (37)$$

In Equation (37), M_D is the total moment at D, $\sum_D^H p\ X$ is the vertical shear at D, $\sum_P^H p\ Y$ is the horizontal shear at D, X is the vertical load increment between D and F, and $p\ Y$ is the corresponding horizontal load increment. In Table 5, j and k are represented by X and Y, respectively, and n and m by Δx and Δy.

Accordingly, Equation (37) may be revised as follows:

$$M_F = M_D + \Delta x \left(\frac{1}{2}\, p\ X + \sum_D^H p\ X\right) + \Delta y \left(\frac{1}{2}\, p\ Y + \sum_D^H p\ Y\right). \quad (38)$$

Evidently, $\frac{1}{2}\, p\ X + \sum_D^H p\ X$ and $\frac{1}{2}\, p\ Y + \sum_D^H p\ Y$ are, respectively, the vertical and horizontal shear at S, and the moment equation may be further revised to read,

$$M_F = M_D + \Delta x \sum_S^H p\ X + \Delta y \sum_S^H p\ Y \ldots\ldots\ldots (39)$$

To facilitate the application of these formulas a computation form has been devised, as shown in Table 5. The arch used as an example is at Elevation 4 625 of the Gibson Dam, on the Sun River Project. (See, also, Table 4.)

In Table 5, Columns (1) and (4) represent basic data, taken from the drawing of the arch to be analyzed, T being the average thickness of the voussoirs, and α the radial angle from the crown to the center of the various voussoirs. Column (2) is used only in the derivation of the reciprocal of the moment of inertia, Column (3).

Column (5) is used for determining values of x and the vertical components of forces and movements. Column (6) is used for the computation of horizontal components of forces and deflections. Column (7) is for use in the computation of Column (12). Columns (8) and (9) are for use in the production of other columns containing these functions, and for insertion in the equations for determining the crown forces. The co-ordinates, x and y, Columns (10) and (12), are, respectively, the horizontal and vertical distances from the crown to the center of the voussoirs, as indicated in Fig. 14.

For a simple circular center line, x is equal to radius times sine α and y is equal to the radius times vers. α. The values of Δx and Δy, Columns (11) and (13), Table 5, may be obtained by taking the differences of consecutive values in Columns (10) and (12), respectively. For a non-circular, or compound arch center line, the values of the co-ordinates and their differences cannot be computed by the method outlined, and suitable changes in column designations and in the method of procedure will be required. Columns (14) and (15) are used in the preparation of other columns, and their sums are used in the determination of the crown forces.

The load co-ordinates, X and Y, Columns (16) and (17), Table 5, correspond to the differential co-ordinates, Δx and Δy, Columns (11) and (13), except that they are measured, or computed to the up-stream face of the dam. Where the water face and the center line of the arch are not concentric, the computations of these co-ordinates are complicated. They usually may be scaled with sufficient accuracy. The method of laying these co-ordinates out is shown on Fig. 15. The points, B, D, and F, are at the mid-points of the center lines of their respective voussoirs, and the lines, BC, DE, and FG, are radial to the water face of the dam. Obviously, the water load coming on the chord, FD, is included between the points, G and E, and similarly for the other chord lengths.

In Table 5, Column (18) contains one-half the average unit water loads for Divisions AC, CE, etc. The loads may be measured at the two ends of the divisions and averaged; or the measured, or computed, load at the center of each division may be taken as the average. Columns (19) and (20) show, respectively, one-half the vertical and horizontal loads on the voussoirs. Summations of the load components, from the crown toward the abutment, shown in Columns (21) and (22), are used to compute the water-load moments, M_L, and the water load thrusts, H_L, shown in Columns (23) and (24). A description of the method of computing these moments and thrusts will bring out the reasons for using half loads in Columns (18), (19), and (20).

TABLE 5.—COMPUTATIONS FOR LEFT HALF OF ARCH, GIBSON DAM.*

Point.	T.	T^3.	$\frac{1}{I}$.	α.	sin α.	cos α.	vers α.	$\frac{\cos\alpha}{T}$.	$\frac{\cos^2\alpha}{T}$.*	x.	Δx.	y.	Δy.	$\frac{Y}{I}$.	$\frac{y^2}{I}$.†	Load Co-ordinate.	
																X.	Y.
Column No..	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
Crown.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
											14.58		0.30			15.2	0.4
1	36.1	47 046	0.00025507	2° 19′	0.04042	0.99918	0.00082	0.027678	.....	14.58	.....	0.30	.....	0.0000765210	.....	.....	.....
											29.08		2.35			30.5	2.3
2	36.3	47 832	0.00025088	6° 57′	0.12100	0.99265	0.00735	0.027346	.....	43.66	.....	2.65	.....	0.0006648320	.....	.....	.....
											28.78		4.70			30.3	4.7
3	37.0	50 653	0.00023691	11° 35′	0.20079	0.97963	0.02037	0.026476	.....	72.44	.....	7.35	.....	0.0017412885	.....	.....	.....
											28.32		7.01			29.7	6.8
4	38.0	54 872	0.00021869	16° 13′	0.27927	0.96021	0.03979	0.025269	.....	100.76	.....	14.36	.....	0.0031403884	.....	.....	.....
											27.66		9.26			29.2	9.2
5	39.2	60 236	0.00019922	20° 51′	0.35592	0.93451	0.06548	0.023840	.....	128.42	.....	23.62	.....	0.0047055764	.....	.....	.....
											26.81		11.48			28.5	11.3
6	40.8	67 917	0.00017669	25° 29′	0.43025	0.90271	0.09729	0.022125	.....	155.23	.....	35.10	.....	0.0062018190	.....	.....	.....
											25.80		13.60			27.6	13.5
7	42.5	76 765	0.00015632	30° 07′	0.50176	0.86500	0.13499	0.020353	.....	181.03	.....	48.70	.....	0.0076127840	.....	.....	.....
											24.62		15.65			26.6	15.7
8	44.7	89 314	0.00013436	34° 43′	0.57000	0.82165	0.17835	0.018381	.....	205.65	.....	64.35	.....	0.0086460660	.....	.....	.....
											23.28		17.58			25.5	17.7
9	47.0	103 823	0.00011558	39° 23′	0.63450	0.77292	0.22708	0.016445	.....	228.93	.....	81.93	.....	0.0094694694	.....	.....	.....
											21.78		19.40			24.2	19.7
10	49.6	122 024	0.00009834	44° 01′	0.69487	0.71914	0.28086	0.014499	.....	250.71	.....	101.33	.....	0.0099647922	.....	.....	.....
Springing line.....	.....	.....	.....	46° 20′	0.72337	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Sigma.....	.....	.....	0.00184206	.....	.....	.....	.....	.....	0.203097293	.....	.....	.....	.....	0.0522235369	3.101191720	.....	.....

* Right half similar.

† Computed and added by machine.

TABLE 5.—(*Continued.*)

Point.	One-half unit water load $= \frac{1}{2} P$.	Vertical water load $= \frac{1}{2} p X$.	Horizontal water load $= \frac{1}{2} p Y$.	$\Sigma p X$.	$\Sigma p Y$.	$\Sigma (\Delta x \Sigma p X + \Delta y \Sigma p Y) = M_L$.	$\sin \alpha \Sigma p X - \cos \alpha \Sigma p Y = H_L$.	$\frac{M_L}{I}$. *	$\frac{M_L y}{I}$. *	$\frac{H_L \cos \alpha}{T}$. *	$H_C y$.	$M = M_C + H_C y + V_C x - M_L$.	$\frac{H_C}{\cos \alpha}$.	$H = H_L + H_C \cos \alpha - V_C \sin \alpha$.	$\frac{1}{2} \frac{M}{I}$.
Column No..	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)	(27)	(28)	(29)	(30)	(31)	(32)
Crown.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	+5 379 826	.....	604 086	.....
1	1 225	18 620	490	18 620	490	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
				37 240	980	271 627	526	.....	.....	.....	181 226	+5 289 425	603 591	604 117	+674.587
2	1 175	35 838	2 703	73 078	3 683	.....	.....	.....	.....	.....	1 600 828	+4 575 264	599 646	606 486	+573.921
				108 916	6 386	2 405 390	6 840	.....	.....	.....	.....	.....	.....	.....	.....
3	1 100	33 330	5 170	142 246	11 556	.....	.....	.....	.....	.....	4 440 032	+3 266 315	591 781	610 650	+386.911
				175 576	16 726	6 553 543	18 869	.....	.....	.....	.....	.....	.....	.....	.....
4	1 060	31 482	7 208	207 058	23 934	.....	.....	.....	.....	.....	8 674 675	+1 469 298	580 049	616 763	+160.660
				238 540	31 142	12 585 203	36 714	.....	.....	.....	.....	.....	.....	.....	.....
5	925	27 010	8 510	265 550	39 652	.....	.....	.....	.....	.....	14 268 511	− 649 156	564 524	623 645	− 64.662
				292 560	48 162	20 297 493	59 120	.....	.....	.....	.....	.....	.....	.....	.....
6	725	20 663	8 198	313 223	56 355	.....	.....	.....	.....	.....	21 208 419	−2 758 712	545 314	630 700	−243.718
				333 886	64 548	29 341 957	85 386	.....	.....	.....	.....	.....	.....	.....	.....
7	625	17 250	8 438	351 136	72 986	.....	.....	.....	.....	.....	29 418 988	−4 595 062	522 534	636 944	−359.150
				368 386	81 424	39 393 876	114 410	.....	.....	.....	.....	.....	.....	.....	.....
8	550	14 630	8 635	383 016	90 059	.....	.....	.....	.....	.....	38 872 934	−5 980 398	496 347	641 913	−401.768
				397 646	98 694	50 233 153	145 566	.....	.....	.....	.....	.....	.....	.....	.....
9	475	12 113	8 408	409 759	107 102	.....	.....	.....	.....	.....	49 492 766	−6 782 604	466 910	645 306	−391.967
				421 872	115 510	61 655 196	178 398	.....	.....	.....	.....	.....	.....	.....	.....
10	.....	.....	.....	430 280	122 405	.....	.....	.....	.....	.....	61 212 034	−6 809 491	434 422	646 268	−334.823
				438 688	129 300	73 401 351	211 846	.....	.....	.....	.....	.....	.....	.....	.....
Springing line.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
Sigma.....	.....	.....	.....	.....	.....	.....	.....	41 457.477	2 379 526.35	15 937.04	....				

* Computed and added by machine.

TABLE 5.—(*Continued.*)

Point.	$\Sigma \frac{M}{I}$.	$s^2 \Sigma \frac{M}{I}$.	Σ Column (34) $\cos a$.	$-\frac{s}{E} \frac{H}{T}$	$\frac{s}{E} \frac{H}{T} - c\,t\,s.$	M — Column (37) $\sin a$.	$y\,d =$ Column (38) + Column (39).	M — Column (34) $\sin a$.	M — Column (37) $\cos a$.	$x\,d =$ Column (40) + Column (41).	Radial deflection = Column (39) $\cos a$ — Column (42) $\sin a$.	Eccentricity = M (Column (29)) H Column (31).	$\frac{\text{Column (31)}}{144\,T}$.	$\frac{\text{Column (32)}\ T}{144}$.	Stress, in Pounds per Square Inch. Extrados, Column (45) + Column (46).	Stress, in Pounds per Square Inch. Intrados, Column (45) — Column (46).
Column No..	(33)	(34)	(35)	(36)	(37)	(38)	(39)	(40)	(41)	(42)	(43)	(44)	(45)	(46)		
Crown.....	.....	.....	.....	.....	.....	.....	0.067867	.....	.....	—0.00001	—0.008	+ 8.91	.....	+162	+278	— 46
1	— 674.595	—0.001994	0.058418	0.001697	0.002677	0.009454	0.029803	—0.022793	—0.022793	—0.002604	—0.065	+ 8.76	+116	+145	+261	— 29
2	—1 923.103	—0.005684	0.056420	0.001693	0.002673	0.009346	0.065766	0.027232	—0.020118	—0.004560	—0.058	+ 7.54	+116	+145	+261	— 29
3	—2 883.935	—0.008524	0.050778	0.001650	0.002630	0.009028	0.056801	0.022084	—0.017465	—0.005495	—0.047	+ 5.35	+113	+ 42	+155	+ 71
4	—3 431.506	—0.010148	0.042428	0.001645	0.002625	0.008495	0.050925	0.020323	—0.014688	—0.005122	—0.036	+ 2.88	.....	.....	.....	.....
5	—3 527.501	—0.010427	0.033689	0.001612	0.002592	0.007761	0.040456	0.017490	—0.012398	—0.003833	—0.025	— 1.05	+107	— 69	+ 38	+176
6	—3 219.124	—0.009515	0.022945	0.001565	0.002545	0.006839	0.029784	0.013779	—0.009946	—0.002087	—0.016	— 4.37	.....	.....	.....	.....
7	—2 616.256	—0.007733	0.014332	0.001518	0.002498	0.005744	0.020076	0.009685	—0.007648	—0.000318	—0.010	— 7.21	+100	—124	— 24	+224
8	—1 855.343	—0.005484	0.007648	0.001455	0.002435	0.004491	0.012134	0.005805	—0.005487	+0.000808	—0.005	— 9.32	.....	.....	.....	.....
9	—1 061.613	—0.003138	0.003187	0.001891	0.002871	0.003108	0.006240	0.002679	—0.003487	+0.000966	—0.002	—10.51	+90	—115	— 25	+205
10	— 384.823	—0.000990	0.000712	0.001920	0.002900	0.001598	0.002310	0.000688	—0.001654	.....	.....	.....	.....	.....	.....	.....
Springing line.....																
Sigma.																

In order that the moments at the various points of diversion may be computed progressively, use is made of the rule that the moment at any point is equal to the moment at a preceding point, plus the shear at the preceding point, times the distance between the two points, plus the moment of any intervening loads.

If only the moments were desired the load co-ordinates might be measured, therefore, between the ends, rather than the centers of the voussoirs, and the full water loads used. However, the water thrusts are functions of the shears at the centers of the voussoirs. For this reason the water loads are calculated and summed by halves, giving the load summations at both the centers and the ends of the voussoirs. Additional columns might be provided between Columns (22) and (23), in which to record and combine $\Delta x \Sigma p X$ and $\Delta y \Sigma p Y$ before summing. Table 5, as shown, is designed for use in connection with a computing machine, and no advantage results from recording intermediate steps. The only disadvantage in this plan is that it causes some confusion as to where the computed sums shall be recorded. It will be noted that two horizontal lines are provided in Table 5 for each voussoir. Functions that apply at the centers of the voussoirs are recorded opposite the numerals 1, 2, 3, etc., in the "point" column. These numerals represent the numbers of the voussoirs. Functions pertaining to the joints between voussoirs are recorded in the intermediate horizontal lines. The increment in moment to any voussoir center is obtained by multiplying the shear at the preceding voussoir end by the preceding value of Δx or Δy, and, if recorded, would be set down in the horizontal line intermediate between the two points to which it applies. However, when summated, these differences become the moments at the points, and, for clearness, should be dropped a half space when recorded in Column (23). The first moment recorded in Column (23), which comes at the center of the first voussoir, is obtained as follows:

$$18\,620 \times 14.58 + 490 \times 0.30 = 271\,627$$

The second moment, which is at the center of the second voussoir, is obtained by adding

$$73\,078 \times 29.08 + 3\,683 \times 2.35$$

to the previous moment; and the remainder are computed in a similar manner to the center of the last voussoir. Column (23), therefore, may be produced by multiplying, successively, in horizontal pairs, the voussoir joint values in Columns (21) and (22) by the corresponding values in Columns (11) and (13), recording a half space down, in Column (13), the sum of the products after the completion of each pair of multiplications. The computing machine must not be cleared between multiplications.

The thrusts, H_L, in Column (24) are obtained from the figures in Columns (21) and (22), as indicated, without continuous summation, the values being recorded opposite the numbers from which they are derived.

The derivation of Columns (25) to (27), inclusive, is evident. The summations of these columns complete the data required for the determination of the crown forces, from Equations (27), (28), and (29). A convenient form for use in the computation of the constants in these equations is arranged at

the end of Table 3. Space is also provided in which the computations for M_C, H_C, and V_C may be made.

Having obtained the crown forces, the moments and thrusts at intermediate points are readily computed, as shown in Columns (28) to (31), inclusive.

The remaining columns of Table 5 are useful in the determination of deflections and stresses. The method of computing the deflections may be explained by reference to Fig. 16. Because of the bending in the voussoir, AC, the tangent to the center line at C will be deflected from its original direction through a small angle, the value of which is,

$$\Delta \beta_B = \sum_C^A \Delta s \frac{M}{E I} \quad \ldots\ldots (40)$$

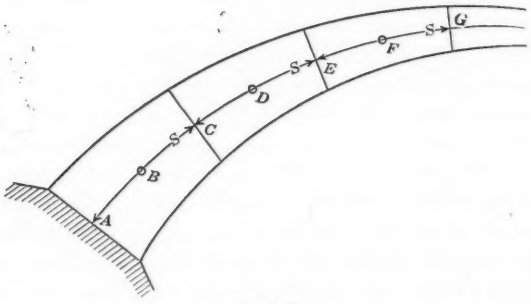

FIG. 16.

If the values of M and I as computed in Table 5 for the point, B, are taken as the average values for the voussoirs, Equation (40) may be written thus:

$$\Delta \beta_B = s \frac{M_B}{E I_B} \quad \ldots\ldots (41)$$

This bending will continue through the succeeding voussoirs, the increments for the voussoirs with centers at D and F being, respectively,

$$\Delta \beta_B = s \frac{M_D}{E I_D} \quad \ldots\ldots (42)$$

$$\Delta \beta_F = s \frac{M_F}{E I_F} \quad \ldots\ldots (43)$$

The total angular movement at E, the upper end of Voussoir D, is, therefore,

$$\beta_E = s \frac{M_B}{E I_B} + s \frac{M_D}{E I_D} = \sum_E^A s \frac{M}{E I} \quad \ldots\ldots (44)$$

The movement at G is, likewise,

$$\beta_E = s \frac{M_B}{E I_B} + s \frac{M_D}{E I_D} + s \frac{M_F}{E I_F} = \sum_F^A s \frac{M}{E I} \quad \ldots\ldots (45)$$

The average angular movement for any voussoir may be taken as the angular movement at its center. Therefore, in Fig. 16, the summations should

be made to the points, B, D, F, etc., if the average angles are desired. In each case, such a summation will include only one-half the last ΔB. For this reason half values of $\frac{M}{I}$, Column (33), is obtained by adding each half value in Column (32) twice, reading the summation between the first and last additions of each half value. The factor, s, is omitted from Column (33), to be introduced later, in Column (34). This is possible only when s is constant throughout the length over which the summation is made. The modulus of elasticity, E, is likewise omitted from this column.

These angular movements cause the points along the arch center to be displaced in space. The point, C, will move with reference to A, a distance,

$$\delta_C = \frac{1}{2} s \beta_C = \frac{1}{2} s^2 \frac{M_B}{E I_B} \quad \text{.....(46)}$$

The direction of motion will be approximately parallel to the radius at B.

The point, E, will move with reference to C, approximately parallel to the radius at D, a distance,

$$\delta_E = s \beta_C + \frac{1}{2} s \Delta \beta_D = \sum_D^A s^2 \frac{M}{E I} \quad \text{.....(47)}$$

It should be noted that δ_E is the movement of E with reference to C. The movement of E with respect to A will be the geometric sum of δ_C and δ_E. This sum can be most readily obtained by resolving δ_C and δ_E into horizontal and vertical components and adding the components. In the example, Table 5, the individual movements are shown in Column (34). The vertical components of these individual movements are obtained by multiplying each term in Column (34) by the corresponding value of cos α, as recorded in Column (6). The individual components are not recorded in Table 5. The summed products are shown in Column (35); the multiplications are made successively, beginning at the bottom of the column, without clearing the computing machine; and the total is recorded after each operation a half space above the last factors used. The values shown in Column (35) are the vertical deflections of the upper ends of the voussoirs, due to bending only. The horizontal deflections, due to bending, shown in Column (40) are computed in the same way, substituting — sin α for cos α. The minus sign is required in Column (40) for the left half of the arch because of the negative horizontal movement.

In addition to the deflections caused by bending, points on the arch center line are displaced due to changes in length of the voussoirs. The amount that each voussoir will shorten, due to thrust, is shown in Column (36) and the net shortening due to thrust and temperature is shown in Column (37). The values in Column (37) multiplied by — sin α and summed, from the abutment toward the crown, as shown in Column (38), give the vertical deflections due to change in length of the arch center line. In the same way, using — cos α, the horizontal deflections are derived, as shown in Column (41). The negative cosines are required because a positive thrust and a negative temperature change produce negative horizontal and vertical deflections for the left half of the arch.

The total y-deflections, obtained by adding Columns (35) and (38), are shown in Column (39); and the total x-movements, the sum of Columns (40) and (41), are shown in Column (42). These component deflections give the complete computed displacement, in space, of points at the ends of the voussoirs, as *C*, *E*, and *G*, Fig. 16. The resultant radial deflections may be obtained as indicated in Column (43).

The eccentricity, Column (44), is obtained by dividing the moment by the thrust, and is required only when the line of pressure is to be platted, or the portion of the section subject to tension determined. Columns (45) and (46) show, respectively, the average direct pressure and the unit fiber stress due to moment. Both columns are divided by 144 to convert the stresses into pounds per square inch.

The final maximum and minimum fiber stresses are shown in the two columns, which are derived from Columns (45) and (46) as indicated. This numerical example refers to a symmetrical case. Table 5 may be extended to unsymmetrical arch dams by inserting the following columns: $(7a)$, $\frac{\sin \alpha}{T}$; $(8a)$, $\frac{\sin^2 \alpha}{T}$; $(9a)$, $\frac{\sin \alpha \cos \alpha}{T}$; $(13a)$, $\frac{x}{I}$; $(14a)$, $\frac{x^2}{I}$; $(15a)$, $\frac{x\,y}{I}$; $(25a)$, $\frac{M_L\,x}{I}$; $(26a)$, $\frac{H_L \sin \alpha}{T}$; $(28a)$, $V_C\,x$; and $(30a)$, $V_C \sin \alpha$.

The constants required in the computations may be determined as shown in Table 6, the subscripts, *L*, denoting the left half of the arch, and *R*, denoting the right half. Use arch data values, as follows: For the left half: $s = 29.176785$ ft.; $l = 260.992$ ft.; R (up stream) $= 405.00$ ft.; and $t = -6°$. For the right half: $s = 29.176785$ ft.; $l = 260.992$ ft.; R (crown) $= 360.8$ ft.; $c = 0.0000056$; and $E = 288\,000\,000$. Then,

$$c\,t\,E = 0.0000056 \times 288\,000\,000 \times 6 = 9\,676.8$$

$$\frac{c\,t\,E\,l_L}{s} = \frac{0.0000056 \times 6 \times 288\,000\,000 \times 260.992}{29.176785} = -86\,560.87$$

$$\frac{s^2}{E} = \frac{29.176785^2}{288\,000\,000} = 0.0000029558$$

$$\frac{s}{E} = \frac{29.176785}{288\,000\,000} = 0.0000001013$$

$$c\,t\,s = 0.0000056 \times 6 \times 29.176785 = 0.00098$$

Use Table 6 to solve the following simplified equations:

$$A_1M_c + B_1H_c + C_1V_c = D_1 \quad \ldots\ldots\ldots\ldots\ldots\ldots (48)$$

$$C_1M_c + B_2H_c + C_2V_c = D_2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (49)$$

$$B_1M_c + B_3H_c + B_2V_c = D_3 \quad \ldots\ldots\ldots\ldots\ldots\ldots (50)$$

Example.—

By Equation (48):

$$0.00184206\,M_c + 0.052223537\,H_c + 0 = 41\,457.477$$

By Equation (50):

$$0.052223537\,M_c + 3.30428902\,H_c + 0 = 2\,277\,028.44$$

TABLE 6.—COMPUTATIONS FOR CONSTANTS, GIBSON DAM.

$\Sigma_L \frac{s}{I}$	0.00184206	$\Sigma_L \frac{M_L x s}{I}$	
$+\Sigma_R \frac{s}{I}$		$-\Sigma_R \frac{M_R x s}{I}$	
$= A_1$	0.00184206	$+\Sigma_L \frac{H_L s \sin \alpha}{T}$	
$\Sigma_L \frac{s y}{I}$	0.052223537	$-\Sigma_R \frac{H_R s \sin \alpha}{T}$	
$+\Sigma_R \frac{s y}{I}$		$-c\ t\ E\ (h_L - h_R)$	
$= B_1$	0.052223537	$= D_2$	
$\Sigma_L \frac{s x}{I}$		$\Sigma_L \frac{s y^2}{I}$	3.101191720
$-\Sigma_R \frac{s x}{I}$		$+\Sigma_R \frac{s y^2}{I}$	
$= C_1$		$+\Sigma_L \frac{s \cos^2 \alpha}{T}$	0.203097293
$\Sigma_L \frac{M_L s}{I}$	41 457.477	$+\Sigma_R \frac{s \cos^2 \alpha}{T}$	
$+\Sigma_R \frac{M_R s}{I}$		$= B_3$	3 304 289.02
$= D_1$	41 457.477	$\Sigma_L \frac{M_L y s}{I}$	2 379 526.35
$\Sigma_L \frac{x y s}{I}$		$+\Sigma_R \frac{M_R s y}{I}$	
$-\Sigma_R \frac{x y s}{I}$		$-\Sigma_L \frac{s H_L \cos \alpha}{T}$	−15 937.04
$-\Sigma_L \frac{s \sin \alpha \cos \alpha}{T}$		$-\Sigma_R \frac{s H_R \cos \alpha}{T}$	
$+\Sigma_R \frac{s \sin \alpha \cos \alpha}{T}$		$+\frac{c\ t\ E\ (l_L + l_R)}{s}$	−86 560.87
$= B_2$		$= D_3$	2 277 028.44
$\Sigma_L \frac{s x^2}{I}$		$H_C = 604.086$	
$+\Sigma_R \frac{s x^2}{I}$		$M_C = +5\,379.826$	
$+\Sigma_L \frac{s \sin^2 \alpha}{T}$		$r_C = 0$	
$+\Sigma_R \frac{s \sin^2 \alpha}{T}$			
$= C_2$			

Solving simultaneously and eliminating M_c,

$$H_c = 604\,086 \text{ lb.}$$

and,

$$M_c = 5\,379\,826 \text{ ft-lb.}$$

The constants given in Table 6 without arithmetical values will be needed in some of the problems encountered.

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STREAM FLOW IN GENERAL TERMS

By Melvin D. Casler,* M. Am. Soc. C. E.

Synopsis

The purpose of this paper is to present a workable general method for the analysis of stream flow in irregular channels—channels in which the invert slope and the channel cross-section are not constant. To attempt to apply the Chezy formula to such a case is to force a perfectly good formula into a situation to which it is not properly applicable. Various scattered attempts have been made to treat this question as applied to special cases, but no satisfactory general treatment has been found in such a form as to be readily followed by most computers.

An exhaustive consideration of the subject of "back-water curves" is contained in "Technical Reports," Part III, of The Miami Conservancy District, but this discussion is confined to "rectangular channels the width of which is uniform and so great in comparison with the depth that the hydraulic radius may be considered equal to the depth".

The discussion contained in this paper is general, and demonstrations of its applicability to weirs, orifices, and siphons are appended.

The Chezy formula, $v = C\sqrt{r\,s}$, is the one most commonly used in stream-flow computations. The factor of longitudinal slope, s, is sometimes referred to as the slope of the water surface and sometimes as the slope of the bed of the stream. As a matter of fact, it is neither of these except under special conditions. The Chezy slope factor should be defined as the friction factor, or rate of loss of head. This is equal to the slope of the water surface only when the velocity is constant at successive cross-sections and is equal to the slope of the stream bed only when "depth of water plus velocity head" is constant, as demonstrated subsequently.

Note.—Written discussion on this paper will be closed in May, 1928.

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In the following discussion, the word, "channel", applies to any form of watercourse—canal, conduit, flume, pipe, or stream valley—flowing partly full, or full, or under pressure; and the formulas presented are applicable to all the cases mentioned.

Nomenclature

The following nomenclature has been used:

Let w = unit weight of water, in pounds per cubic foot.

Q = quantity of flow, in cubic feet per second.

W = quantity of flow, in pounds per second = $w\,Q$.

a = cross-sectional area of stream, in square feet.

d = maximum depth of water from channel invert to free water surface at the section considered (measured to hydraulic gradient in case of full conduit), in feet.

b = width of channel at water surface (= zero in case of full conduit), in feet.

p = wetted perimeter, in feet.

r = hydraulic radius $\left(=\frac{a}{p}\right)$, in feet.

v = mean velocity $\left(=\frac{Q}{a}\right)$, in feet per second.

g = gravity factor (= 32.16).

C = Chezy coefficient.

$m = \frac{1}{2\,g\,a^2}$.

$n = \frac{P}{a^3\,C^2}$.

h = effective velocity head $\left(=\frac{v^2}{2\,g}=\frac{Q^2}{2\,g\,a^2}=m\,Q^2$ if the velocity is uniform throughout the area, $a\right)$.

i = invert drop in direction of flow per running foot of channel, in feet. (If invert rises in direction of flow, i is negative).

s = drop of water surface in direction of flow per running foot of channel, in feet. (If water surface rises in direction of flow, s is negative).

f = lost head per running foot of channel due to friction between the flowing water and the channel perimeter, in feet. ($f = n\,Q^2$, as demonstrated below, and is always positive).

For general treatment, these quantities are to be regarded as coincident factors at one and the same cross-section of the stream.

Let $\int_0^l i\,d\,l$ = total invert drop between two sections l ft. apart ($=I$) ($=i\,l$ if i is constant), in feet.

$\int_0^l s\,d\,l$ = total water-surface drop between two sections l ft. apart ($= S$) ($= s\,l$ if s is constant), in feet.

$\int_0^l f\,d\,l$ = total lost head due to perimeter friction between two sections l ft. apart ($= f\,l$ if f is constant), in feet.

F' = total lost head due to internal friction between two sections l ft. apart (eddies, impact, sudden velocity changes, etc.), in feet.

F = total lost head from all sources between two sections l ft. apart ($= F' + \int_0^l f\,d\,l$), in feet.

F and F' are always positive, while I and S are positive if their gradients drop in the direction of flow, but negative if they rise in that direction.

Evaluation of the Friction Factor, f

The friction factor, f, at any particular section may be evaluated as follows: f is the rate of head loss per foot of channel due to perimeter friction. Hence, $W\,f$ equals the energy in foot-pounds lost, or annihilated, as W moves forward 1 ft., $W = w\,Q = w\,a\,v$ and, therefore, the lost energy $= w\,a\,v\,f$ (Criterion (1)).

This energy loss may be analyzed as follows: $Q = a\,v$ and, hence, v is the length of channel occupied by the quantity of water, Q, or W; and the area of contact between W and the interior of the channel is $p\,v$.

Experiments have demonstrated that the unit frictional resistance is approximately proportional to the square of the mean velocity; that is, the unit frictional resistance, in pounds per square foot of water contact, acting in a direction opposite to that of the stream flow, is Kv^2, K being a constant,

Hence, the total frictional resistance acting against the flow of the quantity of water, W, is $(p\,v) \times (K\,v^2) = K\,p\,v^3$ lb.; and, the energy, in foot-pounds consumed in overcoming this resistance as W moves forward 1 ft., is $K\,p\,v^3$ (Criterion (2)).

From Criteria (1) and (2):

$$K\,p\,v^3 = w\,a\,v\,f,\ v^2 = \frac{w}{K}\,\frac{a}{p}\,f,\ and\ v = C\sqrt{r\,f} \quad \ldots\ldots\ldots (1)$$

Equation (1) is the Chezy formula in general terms, in which, f is the friction factor, $r = \frac{a}{p}$, and $C = \sqrt{\frac{w}{K}}$ = the Chezy coefficient.

$$K = \frac{w}{C^2} = \frac{62.5}{C^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

It is to be noted that the Chezy formula regards the flow as being governed wholly by the perimeter friction.

From Equation (1),

$$f = \frac{p}{a}\,\frac{v^2}{C^2} = \frac{p}{a^3\,C^2}\,Q^2 = n\,Q^2 \quad \ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Since $h = m\ Q^2$ and $f = n\ Q^2$, then,

$$f = \frac{n}{m}\ h \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (4)$$

$$h = \frac{m}{n}\ f \left(\frac{m}{n} = \frac{1}{2\ g\ a^2} \frac{a^3\ C^2}{p} = \frac{a\ C^2}{2\ p\ g} = \frac{C^2}{2\ g}\ r\right) \ldots\ldots\ldots\ldots (5)$$

Steady Non-Uniform Flow

Fig. 1 represents a general case of "steady flow" in an open channel in which the velocity, v, is variable and the longitudinal slopes of the water surface and bed of channel are not parallel. $x\ x$ and $y\ y$ are any two cross-sections l ft. apart, y being down stream from x. The assumption of "steady flow" means that the same quantity of water per second is passing every cross-section of the channel; that is, $Q_x = Q_y$ and $W_x = W_y$.

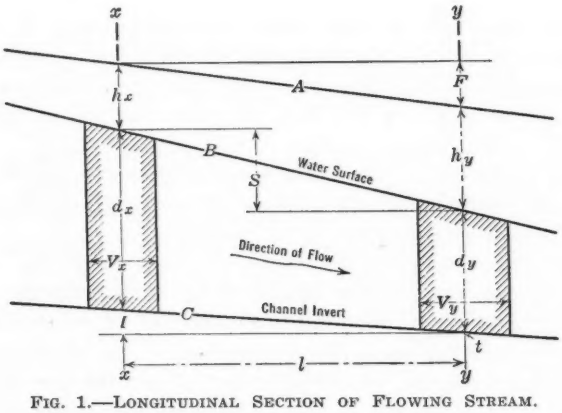

Fig. 1.—Longitudinal Section of Flowing Stream.

The quantity of water, W, occupies the position indicated by the hatched area at Section $x\ x$; and in flowing from x to y, it moves to the position shown by the hatched area at Section $y\ y$. The total energy possessed by this quantity of water, W, with respect to a common datum through the low point, t, is less at y than at x by the amount of energy lost in friction between x and y. Since F is the total head loss, $W\ F$ equals the lost energy.

The static heads with respect to Point t are $(d_x + I)$ at x and d_y at y, and hence the potential energies with respect to t are $W\ (d_x + I)$ at x and $W\ d_y$ at y. The kinetic energy at $x = \frac{W\ v_x^2}{2\ g} = W\ h_x$. The kinetic energy at $y = \frac{W\ v_y^2}{2\ g} = W\ h_y$.

Hence,

$$W\ (d_x + I) + W\ h_x = W\ d_y + W\ h_y + W\ F$$

or,

$$d_x + h_x + I = d_y + h_y + F \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

This relation is shown graphically in Fig. 1, from which it is also apparent that,

$$d_x + I = d_y + S \text{ and } h_x + S = h_y + F \ldots\ldots\ldots\ldots (7)$$

The quantity, $(d + h)$, is a measure of the energy available at any section and Lines (A), (B), and (C) of Fig. 1 may be designated respectively, "energy gradient", "hydraulic gradient", and "invert gradient". Line (A) always drops in the direction of flow. Lines (B) and (C) usually do the same, but either or both of them may, under certain local conditions, rise in the direction of flow.

As a general case, Lines (A), (B), and (C), would not be straight lines. They may, however, be considered as straight for a short elemental length, $d\ l$; and in that length, $I = i\ d\ l$, $S = s\ d\ l$, and $F = f\ d\ l$ (making the Chezy formula assumption that the perimeter friction is the only source of loss).

Then, from Equations (6) and (7):

$$(d_x + h_x) - (d_y + h_y) = d\ l\ (f - i) \ldots\ldots\ldots\ldots (8)$$

$$h_x - h_y = d\ l\ (f - s) \ldots\ldots\ldots\ldots (9)$$

From Equation (1), $v = C\sqrt{r\ f}$. Hence the formula, $v = C\sqrt{r\ s}$ can be true only if $f = s$, and from Equation (9), this can be true only if h (and, hence, v) is constant. Likewise, if $f = i$, then $v = C\sqrt{r\ i}$ and from Equation (8), it is seen that in this case $(d + h)$ is constant.

In Equation (6), group the unknown quantities in the left-hand member and the known quantities on the right, as follows:

$$(d_x + h_x) = (d_y + h_y) + (F - I) \ldots\ldots\ldots\ldots (10)$$

if proceeding up stream, or

$$(d_y + h_y) = (d_x + h_x) + (I - F) \ldots\ldots\ldots\ldots (11)$$

if proceeding down stream.

Equations (10) and (11) are applicable to any invert profile and to any shape of channel section, constant or variable, open or closed, flowing freely or under pressure. The stretches, l, between Sections x and y should be taken sufficiently short to produce the desired degree of accuracy and should terminate at all sudden changes in channel section or invert slope. In case of a sudden head loss, sudden change of section, or a vertical rise or drop in invert elevation, consecutive values of $(d + h)$ may be computed by taking l as zero.

From Equations (10) and (11), the energy gradient and the hydraulic gradient (Lines (A) and (B) of Fig. 1) can be computed for any given channel for an assumed Q.

In proceeding up stream, $(d_x + h_x)$, as computed from Equation (10), becomes $(d_y + h_y)$ for the next up-stream stretch of length, l; and in proceeding down stream, $(d_y + h_y)$, as computed from Equation (11), becomes $(d_x + h_x)$ for the next down-stream stretch.

In order to start this procedure, however, a section must be found in which, d and h are known for the assumed Q, and the only place where these quantities are independently known in the case of non-uniform flow is at the "critical section" discussed later under that heading.

Equations (10) and (11) cannot be solved directly for the unknown quantities in the left-hand members. In the first place the term, F, in the right-hand members can be known only approximately until the left-hand members are evaluated. Furthermore, at every section except the "critical section" there will be two possible mathematical values of both d and h for any particular value of $(d + h)$ for a given Q; one combination of d and h giving a small stream section flowing at a high velocity, and the other a larger stream section flowing at a slower velocity.

This may be graphically illustrated as follows: Assume for simplicity a rectangular channel, 10 ft. wide. The coincident values of d and $(d + h)$ for flows of 100 and 200 sec-ft. are computed in Table 1 and plotted in Fig. 2. Assume a Q of 100 sec-ft. in this channel and $(d + h) = 3$. Then, from Fig. 2, the two possible vaues of d are seen to be 0.84 and 2.80.

TABLE 1.—Q-CURVES FOR RECTANGULAR SECTION OF FIG. 2.

d.	$a = 10\ d$.	$m = \frac{1}{2\,g\,a^2}$.	Q = 100 sec-ft.		Q = 200 sec-ft.	
			$h = m\ Q^2$.	$d + h$.	$h = m\ Q^2$	$d + h$.
6.0	60	0.0000043	0.04	6.04	0.17	6.17
4.0	40	0.0000097	0.10	4.10	0.39	4.39
3.0	30	0.0000173	0.17	3.17	0.69	3.69
2.0	20	0.0000389	0.39	2.39	1.55	3.55
1.0	10	0.0001555	1.55	2.55	6.22	7.22
0.5	5	0.0006219	6.22	6.72	24.88	25.38

The general criterion for selecting the correct value of d as between these two mathematical possibilities is as follows: The water will flow at such a depth, d, that the lost head, F, between any two sections will be as nearly as possible equal to the invert drop, I, in that same stretch of channel. Referring to Equations (10) and (11), it is seen that this criterion may be expressed as follows: Successive values of $(d + h)$ will be as nearly as possible equal to each other (Criterion (3)). The application of this criterion is more fully discussed elsewhere in this paper.

The computation of the hydraulic gradient is thus a matter of locating the "critical section" and, with that as a starting point, applying Equation (10) or (11) as guided by Criterion (3).

THE CRITICAL SECTION

The "critical section" of a flowing stream is the "neck of the bottle" and controls the flow of the entire stream. The "critical section" is delivering the maximum, Q, of which it is capable for the particular value of $(d + h)$ under which it is operating. In other words, the "critical section" is flowing at "maximum efficiency" and is, in general, the only section doing so.

It is apparent from Fig. 2 that for any particular value of $(d + h)$ in a known section, as d increases from zero to $(d + h)$, h decreases from $(d + h)$ to zero and Q increases from zero to a certain maximum and again decreases to

zero. For instance, it is seen that in a rectangular section, 10 ft. wide, 200 sec-ft. is the maximum flow that can be delivered with $(d + h)$ at a value of 3.48; or, conversely, 3.48 is the minimum value of $(d + h)$ which will deliver 200 sec-ft. It is also to be noted that for a $(d + h)$ of 3.48 the depth of flow, d, which will deliver the maximum, Q, is 2.32. Therefore, 2.32 is the "critical depth" for a $(d + h)$ of 3.48.

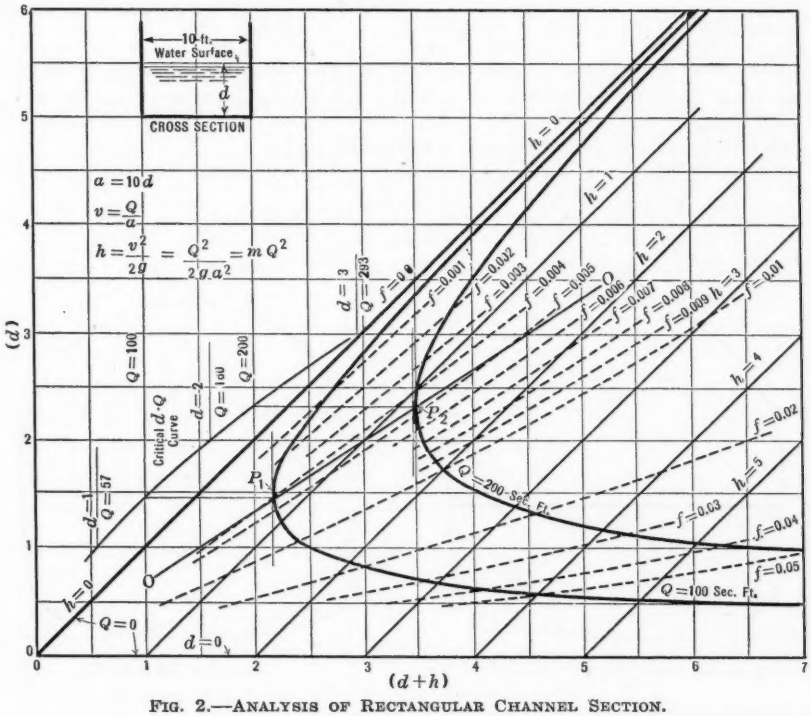

FIG. 2.—ANALYSIS OF RECTANGULAR CHANNEL SECTION.

Hence, for a flow of 200 sec-ft. $(d + h) = 3.48$, $d = 2.32$, and $h = 1.16$, if the section is acting as a "critical section". Otherwise, $(d + h)$ will be greater than 3.48 and d and h will each have two possible values as shown by the 200-curve of Fig. 2, the correct values to be determined by Criterion (3).

A direct method for evaluating d and h, and likewise f, at the "critical section" may be derived in general terms by the aid of calculus, as follows: The value of h to produce a maximum, Q, for a given $(d + h)$ is determined by placing $\frac{d\,Q}{d\,h} = 0$.

Since $Q = a\,v$,

$$d\,Q = a\,(d\,v) + v\,(d\,a) \ldots\ldots\ldots\ldots\ldots\ldots (12)$$

Since $v = \sqrt{2\,g\,h}$,

$$d\,v = \frac{\sqrt{2\,g}}{2\,\sqrt{h}}\,d\,h \ldots\ldots\ldots\ldots\ldots\ldots\ldots (13)$$

(assuming uniform velocity throughout the area, a)

From Equations (12) and (13),

$$d\,Q = \frac{a\sqrt{2\,g}}{2\sqrt{h}}\,d\,h + \sqrt{2\,g\,h}\,d\,a$$

and for maximum, Q,

$$\frac{d\,Q}{d\,h} = \frac{a\sqrt{2\,g}}{2\sqrt{h}} + \sqrt{2\,g\,h}\,\frac{d\,a}{d\,h} = 0 \ldots\ldots\ldots\ldots\ldots\ldots (14)$$

Fig. 3(a) represents a general case of an irregular section, from which it is seen that $d\,a = b\,d\,d = -b\,d\,h$ for a given value of $(d + h)$.

Hence, $\frac{d\,a}{d\,h} = -\,b$ and from Equation (14),

$$\frac{a\sqrt{2\,g}}{2\sqrt{h}} - b\sqrt{2\,g\,h} = 0,\ \frac{a}{2\sqrt{h}} = b\sqrt{h},\ \text{and}\ a = 2\,b\,h \ldots\ldots (15)$$

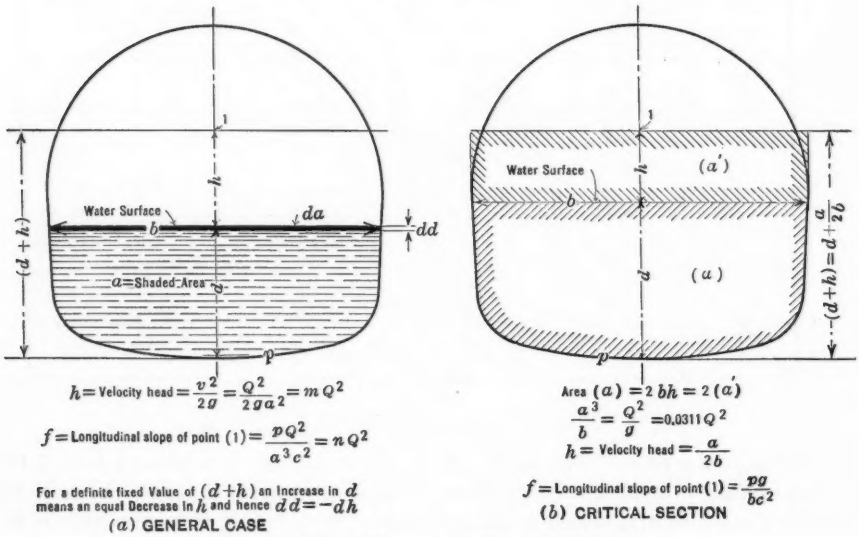

FIG. 3.—CROSS-SECTION OF FLOWING STREAM.

This relation for a maximum, Q, is shown graphically in Fig. 3(b), in which the area, a, is double the area, a'. Hence, at the critical section, from Equation (15):

$$h = \frac{a}{2\,b} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (16)$$

and as $Q = a\,v = a\sqrt{2\,g\,h}$, at the critical section:

$$Q^2 = a^2\,2\,g\,h = \frac{a^3\,g}{b} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (17)$$

$$\frac{a^3}{b} = \frac{Q^2}{g} = 0.0311\,Q^2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots (18)$$

$$Q = 5.671\sqrt{\frac{a^3}{b}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (19)$$

Also,

$$f = n\,Q^2 = \frac{p\,Q^2}{a^3\,C^2} = \frac{p}{a^3\,C^2}\,\frac{a^3\,g}{b}$$

Hence, at the critical section,

$$f = \frac{p\,g}{b\,C^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (20)$$

The three pertinent factors, d, h, and f, at the critical section may now be computed as follows: d is obtained by trial or by direct analysis from Equation (18), which also furnishes the corresponding coincident values of a, b, and p; h is computed from Equation (16); and f is computed from Equation (20). Knowing d, h, and f at the critical section, the values of these factors at other points of the stream are computed progressively from Equation (10) or Equation (11).

If the rectangular section of Fig. 2 is acting as a critical section and $Q = 100$, from Equation (18),

$$\frac{a^3}{b} = 311$$

In this case,

$$\frac{a^3}{b} = b^2\,d^3 = 100\,d^3$$

hence

$$d^3 = 3.11$$

and,

$$d = 1.46$$

From Equation (16),

$$h = \frac{a}{2\,b} = \frac{d}{2} = 0.73$$

$$d + h = 2.19$$

and,

$$p = b + 2\,d = 10 + 2.92 = 12.92$$

From Equation (20),

$$f = \frac{12.92 \times 32.16}{10\ C^2}$$

and, if $C = 100$, $f = 0.0042$

For $Q = 200$, $d^3 = 12.44$; $d = 2.32$; $h = 1.16$; $d + h = 3.48$; and $p = 14.64$; therefore,

$$f = \frac{14.64 \times 32.16}{10 \times 10\,000} = 0.0047$$

The "critical values" of d and $(d + h)$ are located at Points P_1 and P_2 of Fig. 2 and, at these points, the Q-curve tangents must be vertical.

For an irregular section the procedure outlined may become rather complicated. In such a case the critical points, or Q-curve vertices, may be located as follows for the rectangular section of Fig. 2:

1.—Compute a locus of these vertices as determined by the coincident values of (d) and $(d + h)$ of Table 2.

TABLE 2.—CRITICAL FACTORS FOR RECTANGULAR SECTION OF FIG. 2.

d.	a.	b.	h. From Equation (16).	$d + h$.	Q. From Equation (19).
3	30	10	1.5	4.5	295
2	20	10	1.0	3.0	160
1	10	10	0.5	1.5	57

2.—This locus plots as shown by Line OO of Fig. 2. For a rectangular section it is a straight line, but for an irregular section the locus would, in general, be curved.

3.—Plot a critical d-Q-curve from the (d) and (Q)-columns of Table 2 as indicated at the left of Fig. 2.

4.—The intersection of the proper critical (d) line with the locus, OO, will locate the vertex of any desired Q-curve.

It is also a simple matter to plot a set of f-curves crossing the Q-curves of Fig. 2. The f-curves are computed from Equation (5) as illustrated in Table 3 for the rectangular section of Fig. 2.

TABLE 3.—f-CURVES FOR RECTANGULAR SECTION OF FIG. 2.*

d.	a.	p.	$m = \frac{1}{2 g a^2}$.	$n = \frac{p}{a^3 C^2}$.	$\frac{m}{n}$.	h. From Equation (5).
3.0	30	16	0.0000173	0.000000059	293.22	293.22 f.
2.0	20	14	0.0000389	0.000000175	222.29	222.29 f.
1.0	10	12	0.0001555	0.000001200	129.58	129.58 f.
0.5	5	11	0.0006219	0.000008800	70.67	70.67 f.

* C is given a constant value of 100.

It is apparent from Table 3 that for any particular value of d, h varies directly as f and, hence, the $(d + h)$ increments will vary directly as the f increments on the same d line. In other words, equal distances on any horizontal line of Fig. 2 represent equal increments of f.

Hence, the f-curves are easily plotted by graduating the horizontal d lines as follows: For $d = 3$ in Table 3, if $f = 0$, $h = 0$, and $(d + h) = 3.0$; if $f = 0.01$, $h = 2.93$, $(d + h) = 5.93$, and that portion of the $d = 3$ line, between $(d + h) = 3$ and $(d + h) = 5.93$, may be divided into ten equal parts to mark f values of 0.001, 0.002, 0.003, etc. For $d = 1$, if $f = 0.05$,

$h = 6.48$, $(d + h) = 7.48$; and by dividing that portion of the $d = 1$ line, between $(d + h) = 1$ and $(d + h) = 7.48$, into five equal parts, these divisions will be points on the f-curves, 0.01, 0.02, 0.03, 0.04 and 0.05, etc.

The dotted lines of Fig. 2 are plotted in this manner and are seen to check the computed values of $f = 0.0042$ and 0.0047 at Points P_1 and P_2, respectively.

Fig. 4(b) is a chart embodying the principles of Fig. 2 as applied to the conduit of Fig. 4(a). The preparatory computations for Fig. 4(b) are indicated in Table 4 in which the assumed values of C are based on the hydraulic radius, r. The heights from the bottom to the horizontal lines (Fig. 4(a)), are given under d. The dotted critical factor line of Fig 4(b) becomes horizontal at infinity, and for any finite value of h the "critical depth" is less than the maximum vertical depth of the conduit. It is also apparent that when the conduit is flowing full or under pressure, a change in (d) does not alter the area, a, and, consequently, (h) and (f) remain constant for a given Q.

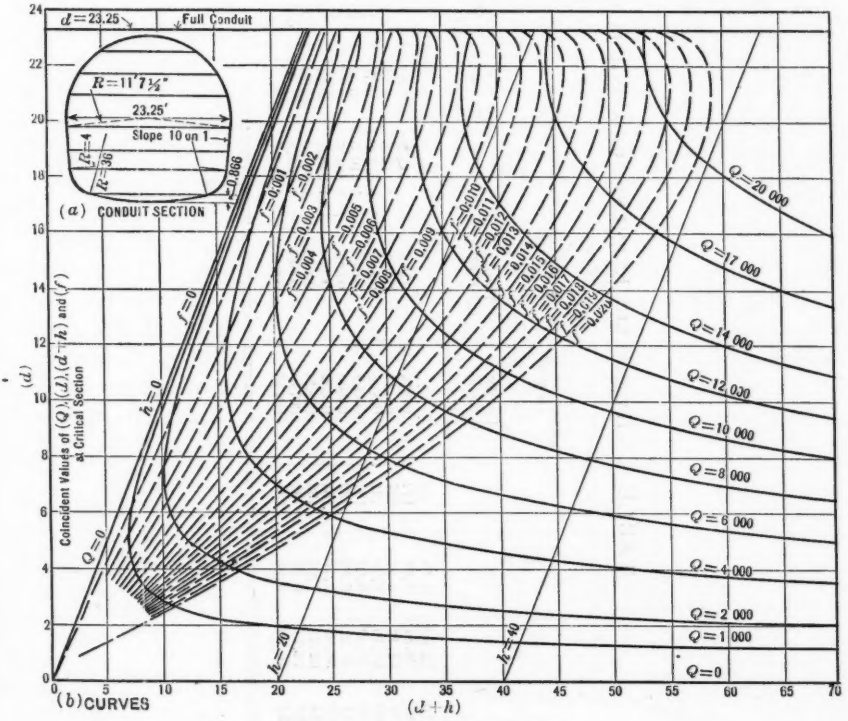

FIG. 4.—ANALYSIS OF CONDUIT.

The computations indicated in Table 4 are, of course, much simplified for a geometrical channel section in which b, a, and p can be expressed as simple algebraic functions of the depth, d.

Location of Critical Section.—The first question which comes up in the application of the foregoing principles is, Where is the "critical section" located?

TABLE 4.—HYDRAULIC ELEMENTS OF CONDUIT OF FIG. 4.

d.	b.	a.	p.	$r = \frac{a}{p}$.	C.	$m^* = \frac{1}{2\,g\,a^2}$.	$n\dagger = \frac{P}{a^3\,C^2}$.	$\frac{m}{n}$.	Critical Factors. (See Dotted Line of Fig. 4(b).)					$f = n\,Q^2$.
						Prefix 0.00000,	Prefix 0.0000000,		$h = \frac{a}{2\,b}$.	$Q^2 = \frac{h}{m}$.	Q.	$v = \frac{Q}{a}$.	$d + h$.	
0.866	15.70	9.10	15.82	0.58	95	.....	.....	80.7	0.29	1 521	39	4.30	1.16	0.0035
4.372	21.93	79.87	25.84	3.09	122	24372	08407	715.3	1.82	748 225	865	10.80	6.19	0.0025
7.000	22.44	138.16	31.12	4.44	127	08144	00782	1 112.6	3.08	8 779 136	1 944	14.10	10.08	0.0028
10.469	23.13	217.20	38.10	5.70	131	03296	00217	1 518.8	4.70	14 250 625	3 775	17.40	15.17	0.0031
11.625	23.25	244.10	40.42	6.04	131	02609	00162	1 610.5	5.25	20 115 225	4 485	18.40	16.87	0.0033
15.000	22.25	321.31	47.27	6.80	132	01506	00083	1 586.6	7.22	47 941 776	6 924	21.55	22.22	0.0039
18.000	19.44	384.52	53.92	7.13	133	01052	00054	1 948.1	9.89	94 051 204	9 698	25.20	27.89	0.0051
21.000	13.75	435.84	62.23	7.00	133	00820	00043	1 907.0	15.83	192 904 321	13 889	31.90	36.83	0.0063
23.000	4.80	455.56	72.11	6.32	132	00749	00044	1 702.3	47.45	633 428 224	25 168	55.25	70.45	0.0279
23.250	0	456.84	76.94	5.93	131	00746	00047	1 587.2	∞	∞	∞	∞	∞	∞

* $m_1 = 0.00018774$.

† $n_1 = 0.0000022836$.

As a general proposition, the channel section may possibly have an entirely different shape at one point than at another and hence the "critical section" may be at different locations for different values of Q, due to relative changes in the hydraulic elements of the various sections at different depths of flow.

The only general criterion for the location of the critical section for a given Q comes from the fact that the value of $(d + h)$ at any section as computed from Equations (10) or (11) cannot be less than the "critical value" of $(d + h)$ for that section for the assumed Q.

Hence, the "critical section" will be that section from which one can start with the critical values of d and h for that section and, by applying progressively Equation (10) or Equation (11), or both, as the case may be, obtain, at all other sections of the stream, values of $(d + h)$ none of which will be less than the critical $(d + h)$ of those respective sections for the assumed Q.

In a channel of varying cross-section, it becomes a matter of selecting a section which judgment would dictate as the probable "critical section". The correctness or incorrectness of this guess will be proven by the progressive application of Equation (10) or Equation (11). If, at any stage of this procedure, a section is encountered at which $(d + h)$, as computed from Equation (10) or Equation (11), is less than the critical $(d + h)$ of that section as computed from Equations (18) and (16), then that section is established as the "critical section" as regards all sections previously computed and a new start is made at that section, the previous computations being revised in reverse order.

These principles are graphically represented in Fig. 5, in which, Line $D\,D$ is the invert profile; Line $K\,K$ is a locus of critical values of $(d + h)$; that is, the vertical intercept between the D-line and the K-line at any section is the critical value of $(d + h)$ at that section for the assumed Q; and Line $L\,L$ is the "energy gradient" and its successive drops represent the friction losses, F, in the various stretches of the channel. Hence, Line $L\,L$ cannot drop below Line $K\,K$ at any point and will be tangent to Line $K\,K$ at the "critical section", as indicated in Fig. 5.

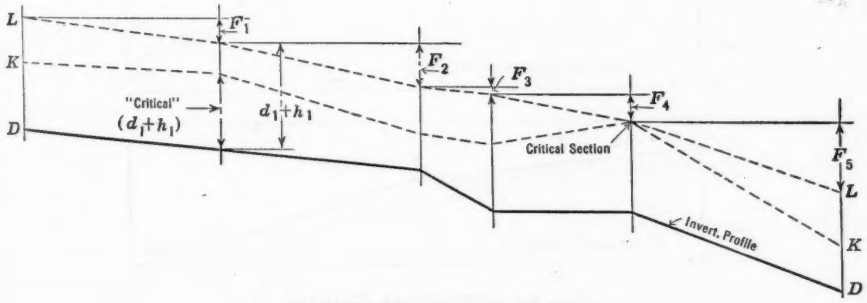

FIG. 5.—CRITICAL SECTION CRITERION.

The only reason the "critical section" cannot be definitely located by this method is that the F-drops are not known until the various water depths are evaluated. These principles may be applied as an aid to judgment, however,

by computing tentative F-values and plotting a trial L L-line and shifting it vertically with its datum kept horizontal in order to locate a point of tangency with Line K K as a first guess on the critical section location.

In many instances, the proposed tentative F-values might logically be based on the supposition that each and every section is acting as a "critical section". The matter of locating the critical section is, of course, much simplified if the channel cross-section is constant, as in the following example.

Illustrative Example.—Let Fig. 4(*a*) represent a conduit of constant cross-section to be analyzed for a flow of 6 000 sec-ft. (see Fig. 6 and Table 5). Suppose the invert slope, i, is constant at 0.002 in the up-stream portion of the conduit, changing abruptly to 0.006 at some point between the inlet and the outlet. In a short elemental length, $d\,l$, assume that $f_x = f_y$ and, considering perimeter friction as the only source of loss, there results from Equation (8),

$$(d_x + h_x) = (d_y + h_y) + d\,l\,(f - i)$$

from which it is evident that $(d + h)$ is increasing down stream if $i > f$, and is decreasing down stream if $i < f$. Also, if the channel section is constant, $(d + h)$ will be less at the "critical section" than at any other section of the stream. Hence, in a channel of constant section, if the critical section is located in a stretch where i is greater than the critical f, it must be at the up-stream end of that stretch; and if in a stretch where i is less than the critical f, it will be at the down-stream end.

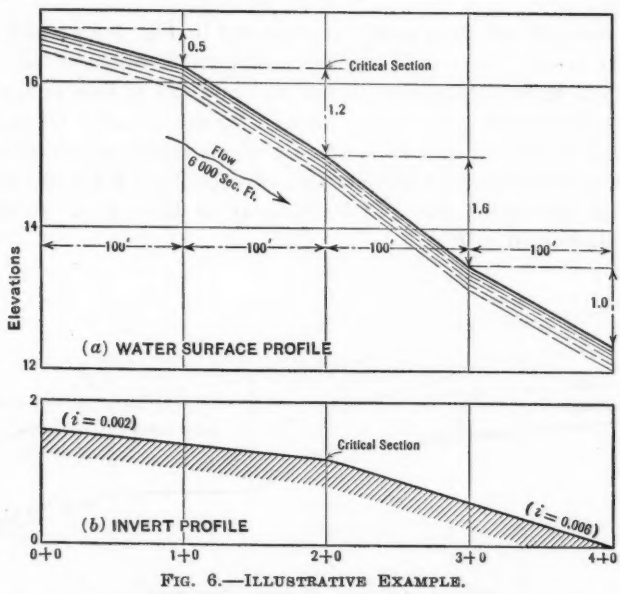

FIG. 6.—ILLUSTRATIVE EXAMPLE.

From Fig. 4(*b*), $f = 0.0037$ at the critical section in the present example ($Q = 6\,000$), and, hence, the critical section must be at the up-stream end of the 0.006 slope, or at the down-stream end of the 0.002 slope. Since these two points coincide, the critical section is definitely located. If the

slopes had been reversed, however, with 0.006 at the up-stream end and 0.002 at the down-stream end, there would have been two possible locations of the critical section to investigate, one at the inlet and one at the outlet, with perhaps the possibility of a critical section at each end and a hydraulic jump within the conduit to bridge the change from low stage to high stage.

TABLE 5.—ANALYSIS OF CONDUIT WITH CONSTANT CROSS-SECTION.*

Station.	$d + h$.	d.	h.	f.	F.	I.	s.	Invert elevation.	Water surface elevation.
0 + 0	20.49	15.1	5.39	0.0029	0.30	0.2	0.5	1.6	16.7
1 + 0	20.39	14.8	5.59	0.0031	0.34	0.2	1.2	1.4	16.2
2 + 0	20.25	13.8	6.45	0.0037	0.41	0.6	1.6	1.2	15.0
3 + 0	20.44	12.8	7.64	0.0045	0.47	0.6	1.0	0.6	13.4
4 + 0	20.57	12.4	8.17	0.0049				0	12.4

*See Fig. 6.

The hydraulic gradient may now be computed by starting at the critical section which, in this case, is between the inlet and the outlet, and proceeding from there both up stream and down stream. Hence, start at the vertex of the $Q = 6\,000$-curve and apply Equation (10) to go up stream, or Equation (11) to go down stream.

Let $I = i\,l$ and assume $F = \left(\frac{f_x + f_y}{2}\right) l$, taking l in 100-ft. stretches. To satisfy Criterion (3), F must be as nearly as possible equal to I or, in the present case $\frac{1}{2}(f_x + f_y)$ must be, as nearly as possible, equal to i.

In proceeding up stream from the critical section, $f_y = 0.0037$ and $i = 0.002$, and it is apparent that one must follow the Q-curve upward from the dotted line of Fig. 6 to make $\frac{1}{2}(f_x + f_y)$ approach the value of i.

Hence, there is the following general criterion in the application of Fig. 4(b), or a similar chart for any constant-section channel, providing f is the only source of head loss: Follow the Q-curve upward or downward from its vertex accordingly as f at the vertex is, respectively, greater or less than i (Criterion (4)).

In this example there is, at the critical section, at the break in invert gradients, $d = 13.8$, $(d + h) = 20.25$, and $f = 0.0037$, as read from the vertex of the $Q = 6\,000$-curve of Fig. 4(b).

Proceeding up stream and following the Q-curve upward from its vertex, per Criterion (4), it is found that, at a point 100 ft. up stream, $(d + h) = 20.39$ and $f = 0.0031$, to satisfy Equation (10) in the following form:

$$(d_x + h_x) - \frac{l}{2} f_x = (d_y + h_y) + \frac{l}{2} f_y - l\,i \quad \ldots\ldots\ldots\ldots (21)$$

$\left(\text{since } F = \left(\frac{f_x + f_y}{2}\right) l \text{ and } I = i\,l\right)$.

From Equation (21),

$$(d_x + h_x) - 50\, f_x = 20.25 + 0.185 - 0.2 = 20.235$$

and,

$$20.39 - (50 \times 0.0031) = 20.235$$

The corresponding water depth, d, as given by Fig. 4(b), is 14.8.

At the up-stream end of the next 100-ft. stretch $(d + h) = 20.49$ and $f = 0.0029$ as follows:

$$(d_x + h_x) - 50\, f_x = 20.39 + (50 \times 0.0031) - 0.2 = 20.345$$
$$20.49 - (50 \times 0.0029) = 20.345$$
$$d = 15.1$$

and so on to the inlet.

Proceeding down stream from the critical section, $i = 0.006$ and, therefore, it must follow the Q-curve downward from its vertex. At a point 100 ft. down stream from the critical section, $(d + h) = 20.44$ and $f = 0.0045$ from the following adaptation of Equation (11):

$$(d_y + h_y) + \frac{l}{2} f_y = (d_x + h_x) - \frac{l}{2} f_x + l\, i \ldots\ldots\ldots\ldots (22)$$

$$(d_y + h_y) + 50\, f_y = 20.25 - 0.185 + 0.6 = 20.665$$
$$20.44 + (50 \times 0.0045) = 20.665$$
$$d = 12.8$$

At the down-stream end of the next 100-ft. stretch, $(d + h) = 20.57$ and $f = 0.0049$ as follows:

$$(d_y + h_y) + 50\, f_y = 20.44 - (50 \times 0.0045) + 0.6 = 20.815$$
$$20.57 + (50 \times 0.0049) = 20.815$$
$$d = 12.4$$

and so on to the outlet.

These results are given in Table 5 and plotted in Fig. 6. In this example, d was computed at points a definite distance apart. This is the method that would be used in the precise analysis of a channel of changing cross-section, plotting curves similar to those of Fig. 4(b) for each section considered.

In a channel of constant cross-section this process can be simplified somewhat by assuming definite values of d and then computing where these depths would occur by solving directly for l instead of assuming l and solving for d by trial and error.

For instance, starting with $d = 13.8$ $(d + h) = 20.25$, and $f = 0.0037$ at the critical section, it is possible to determine directly where $d = 14.8$ as follows: If $d_x = 14.8$, then from Fig. 4(b), $(d_x + h_x) = 20.39$ and $f_x = 0.0031$. Then from Equation (21),

$$l \left(\frac{f_x}{2} + \frac{f_y}{2} - i \right) = (d_x + h_x) - (d_y + h_y)$$

$$l\, (0.00155 + 0.00185 - 0.002) = 20.39 - 20.25$$

$$l = \frac{0.14}{0.0014} = 100$$

If the water level is known at one or more points of the channel, the discharge will be a value of Q which will satisfy the known conditions without resulting in a value of $(d + h)$ and any section less than its critical $(d + h)$ for that Q.

In such a case a situation might be found in which no section of the channel would be operating as a "critical section". If the velocity of approach above the conduit inlet and the velocity of the tail-water below its outlet are so small that they may be neglected, then h at those points becomes zero and Equation (12) becomes,

$$d_x = d_y + F - I, \text{ or } F = d_x - d_y + I$$

in which, d_x and d_y are in the still water of the approach pool and tail-water pool, respectively; I is the total invert drop between those points; and F is the total head loss from all sources.

In this case the discharge would be the maximum value of Q which could be delivered with a total loss of head equal to this value of F, unless the resulting $(d + h)$ at some section should be below its critical value, in which case the discharge would be governed by the "critical section". The analysis, of course, must include the F losses at inlet and outlet or elsewhere due to sudden velocity changes.

If the channel in the foregoing example had not been of constant cross-section it would have been necessary to prepare a chart similar to Fig. 4(*b*) for each of the different channel sections, applying each one in its proper place and observing the general criterion in the location of the "critical section". A sudden change of section would introduce a sudden head loss and a corresponding sudden drop in the value of $(d + h)$.

Weirs, Dams, and Falls

The conditions at the crest of a weir, dam, or falls are usually those of a critical section; that is, the stream section on the crest is delivering the maximum Q possible for the particular value of $(d + h)$ under which it is operating, without hindrance or limitation from other sections up stream or down stream.

Assuming the stream section on the crest to be rectangular, $a = b\,d$ and the following simplifications of the general critical section Equations (16) and (19) occur, representing conditions at the weir crest.

From Equation (16),

$$h = \frac{d}{2} \quad \ldots\ldots (23)$$

and,

$$d + h = 3\ h = \frac{3}{2}\,d \quad \ldots\ldots (24)$$

From Equation (19),

$$Q = 5.671\ b\ d^{\frac{3}{2}} \quad \ldots\ldots (25)$$

and,

$$d = 0.3145 \left(\frac{Q}{b}\right)^{\frac{2}{3}} \quad \ldots\ldots (26)$$

Equation (25) should give the discharge over any form of rectangular weir, dam, or falls, b and d being the exact dimensions of the stream of water on the weir crest.

Because of difficulties in the way of measuring d at the crest, weir discharge is usually expressed in terms of the static head a "short distance up stream" from the weir. This head, usually designated as H, is the difference in elevation between the weir crest and the water surface at the section where H is measured, which corresponds to Section x of Fig 1, and the weir crest corresponds to Section y.

Neglecting friction between x and y, Equation (10), as shown in Fig. 7, becomes,

$$d_x + h_x + I = d_y + h_y \ldots\ldots\ldots\ldots\ldots\ldots (27)$$

In Fig. 7, I is negative since the invert is higher at y than at x. Also, $d_x + I = H$ and from Equation (24), $d_y + h_y = \frac{3\,d_y}{2}$. Hence, from Equation (27),

$$\frac{3\,d_y}{2} = H + h_x$$

and,

$$d_y = \frac{2}{3}(H + h_x) \ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

and from Equations (25) and (28),

$$Q = 3.087\, b\, (H + h_x)^{\frac{3}{2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

In Equation (29), h_x is the effective velocity head at Section x, where H

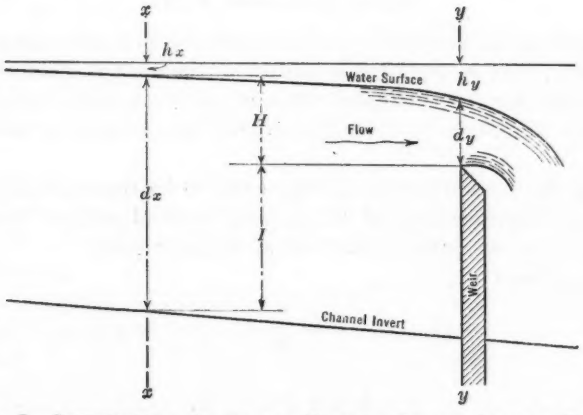

FIG. 7.—LONGITUDINAL SECTION OF FLOWING STREAM ABOVE A WEIR.

is measured, and b is the net width of the stream at the weir crest.

In the Francis weir formula,

$$Q = K\, b\, H^{\frac{3}{2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (30)$$

in which, K is a constant.

From Equations (25) and (30),

$$K H^{\frac{3}{2}} = 5.671\, d_y^{\frac{3}{2}}$$

and,

$$K = 5.671 \left(\frac{d_y}{H}\right)^{\frac{3}{2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (31)$$

It should be kept in mind that these equations assume a uniform velocity throughout the entire area of every cross-section of the stream. $\left(v = \text{the mean velocity}, \frac{Q}{a}, \text{and } h = \frac{v^2}{2g}.\right)$

If the velocity in any cross-section is not uniform, then $h > \frac{v^2}{2g}$. Equation (25) is theoretically correct on the assumption that the velocity on the crest is uniform throughout the area, b d, b and d being the exact dimensions of the stream on the crest.

Equation (29) is theoretically correct on the further assumption of uniform velocity at Section x and no friction losses between x and y.

In Equation (30), if K is determined experimentally it includes the effect of friction losses and non-uniform velocities, but the evaluation of K from Equation (31) as plotted in Fig. 8, involves the same assumptions as Equation (29).

It is not usually feasible to measure d_y with any degree of accuracy, but the principles of Equation (31) may be analyzed in the following manner:

$$b_y\, d_y = a_y \text{ and } a_y\, v_y = a_x\, v_x$$

from which,

$$b_y\, d_y \sqrt{h_y} = a_x \sqrt{h_x}$$

or,

$$b_y^2\, d_y^2\, h_y = a_x^2\, h_x \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (32)$$

From Equation (23), $h_y = \frac{1}{2}\, d_y$ and from Equation (28) $h_x = \frac{3}{2}\, d_y - H$. Hence, from Equation (32),

$$\frac{1}{2}\, b_y^2\, d_y^3 = a_x^2 \left(\frac{3}{2}\, d_y - H\right)$$

or,

$$\left(\frac{a_x}{b_y}\right)^2 = \frac{d_y^3}{3\, d_y - 2\, H} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (33)$$

$5.671 = \sqrt{g}$ and, hence, from Equation (31), $d_y = \frac{H\, K^{\frac{2}{3}}}{g^{\frac{1}{3}}}$. Substituting this value of d_y in Equation (33),

$$\frac{a_x}{b_y} = k\, H \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (34)$$

in which,

$$k = \frac{K}{g^{\frac{1}{3}} \sqrt{3 K^{\frac{2}{3}} - 2 g^{\frac{1}{3}}}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (35)$$

Coincident values of K and k from Equation (35) and corresponding values of $\frac{a_x}{b_y}$ for various values of H are listed in Table 6 and plotted in Fig. 8.

TABLE 6.—WEIR COEFFICIENTS.

K, in Equation (30).	k' from Equation (37).	k from Equation (35).	$\frac{a_x}{b_y}$ FROM EQUATION (34).*			
			$H = 1.$	$H = 2.$	$H = 3.$	$H = 4.$
3.087†	0	∞	∞	∞	∞	∞
3.1	0.0028	7.2458	7.25	14.49	21.74	28.98
3.15	0.0136	3.3679	3.37	6.74	10.10	13.47
3.2	0.0243	2.5617	2.56	5.12	7.69	10.25
3.3	0.0455	1.9293	1.93	3.86	5.79	7.72
3.4	0.0665	1.6437	1.64	3.29	4.93	6.57
3.5	0.0873	1.4768	1.48	2.95	4.43	5.91
3.6	0.1080	1.3663	1.37	2.73	4.10	5.47
3.7	0.1284	1.2877	1.29	2.58	3.86	5.15
3.8	0.1486	1.2292	1.23	2.46	3.69	4.92
4.0	0.1886	1.1486	1.15	2.30	3.45	4.59
4.2	0.2279	1.0971	1.10	2.19	3.29	4.39
4.5	0.2857	1.0498	1.05	2.10	3.15	4.20
5.671‡	0.5000	1.0000	1.00	2.00	3.00	4.00

*See Fig. 8.

†$3.087 = \sqrt{g}\left(\frac{2}{3}\right)^{\frac{3}{2}}$.

‡$5.671 = \sqrt{g}$.

The K lines of Fig. 8 would be straight if plotted to natural scales, but logarithmic plotting is used to reduce the size of the diagram. It is apparent from Fig. 7 that H may have any value between the limits, d_y and $(d_y + h_y)$, depending on the location of Section x.

From Equation (31), when $H = d_y$, $K = 5.671$; and when $H = d_y + h_y \left(= \frac{3}{2} d_y\right)$, $K = 5.671 \left(\frac{2}{3}\right)^{\frac{3}{2}} = 3.087$. Hence, K must lie between the two limits, 5.671 and 3.087.

From Equations (35) and (34), for $K = 5.671 \left(= g^{\frac{1}{2}}\right)$, $\frac{a_x}{b_y} = H$; and for $K = 3.087 \left(= g^{\frac{1}{2}} \times \left(\frac{2}{3}\right)^{\frac{3}{2}}\right)$, $\frac{a_x}{b_y} = \infty$ for all values of H. That is, $a_x = \infty$, or, in other words, $h_x = 0$.

Fig. 8 can now be applied to any weir by plotting on it a curve of coincident values of H and $\frac{a_x}{b_y}$ for that particular weir. Lines $A\,A$ and $B\,B$ of Fig. 8

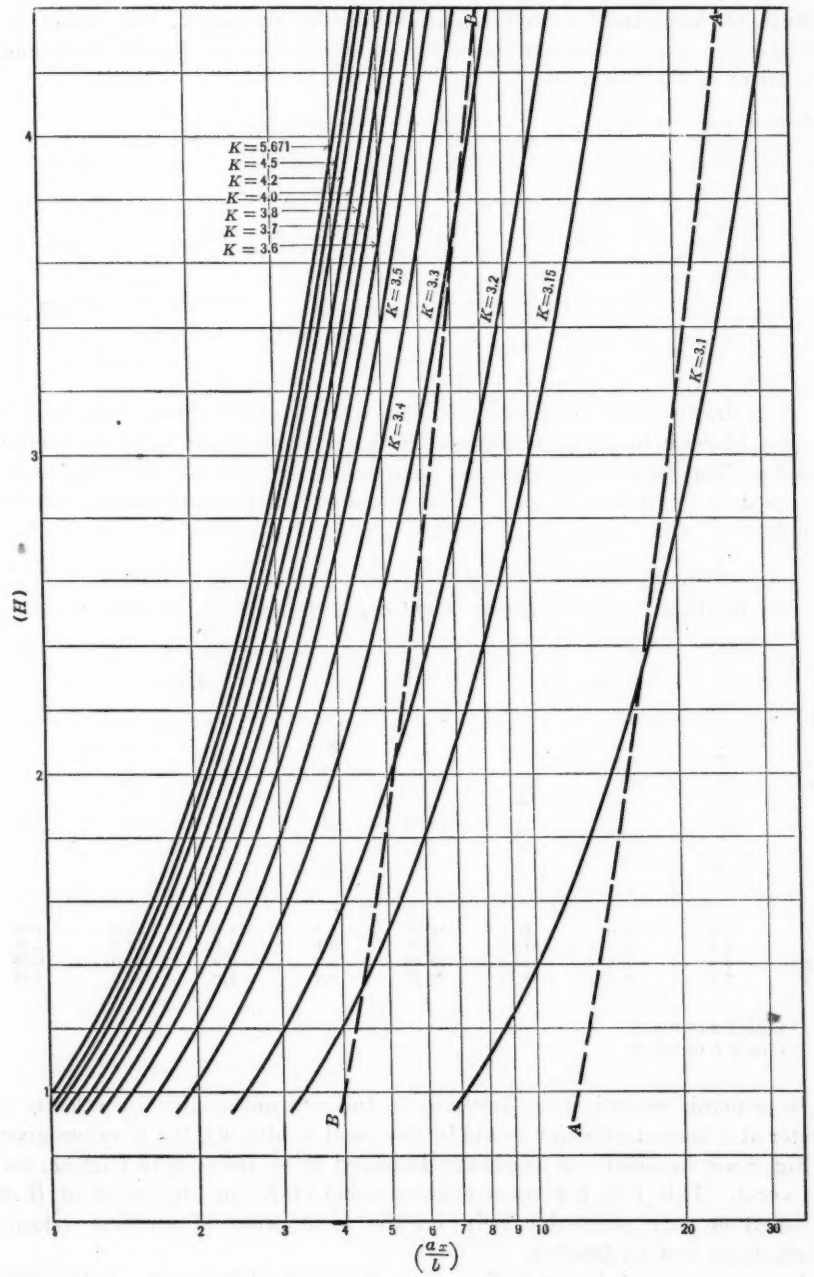

FIG. 8.—VALUES OF k IN $Q = k\,b\,H^{\frac{3}{2}}$, ASSUMING UNIFORM VELOCITIES AND NO FRICTION.

are such curves for the weirs of Fig. 9. Fig. 8 is not put forth as a substitute for experimental determinations of the factor, K, but rather as a groundwork for such determinations and as a means of directly introducing the effect of the factor, a_x.

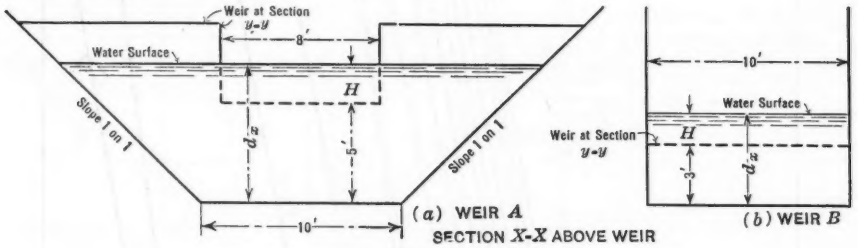

FIG. 9.

It is drawn assuming that a uniform velocity exists throughout the area, a_x and likewise in a_y, and that no frictional losses occur between Sections x and y. The degree to which these assumptions are approached are, of course, governed by the details of the weir crest and of the approach channel between x and y.

TABLE 7.—COINCIDENT VALUES OF H AND $\frac{a_x}{b_y}$ IN FIG. 9.

	WEIR A.				WEIR B.			
H.	$b_y = (8 - 0.2\,H)$.	$d_x = (H + 5)$.	$a_x = d_x\,(10 + d_x)$.	$\frac{a_x}{b_y}$.*	$b_y = (10 - 0.2\,H)$.	$d_x = (H + 3)$.	$a_x = 10\,d_x$.	$\frac{a_x}{b_y}$.†
1	7.8	6.0	96.0	12.31	9.8	4.0	40.0	4.08
2	7.6	7.0	119.0	15.66	9.6	5.0	50.0	5.21
3	7.4	8.0	144.0	19.46	9.4	6.0	60.0	6.38
4	7.2	9.0	171.0	23.75	9.2	7.0	70.0	7.61

* Line $A\,A$ of Fig. 8.
† Line $B\,B$ of Fig. 8.

In general, assuming no friction, if the non-uniformity in velocity is greater at x than at y (which would be the usual condition), the K values given by Fig. 8 are too small. K is always decreased by an increase in friction, and *vice versa*. This Fig. 8 gives a definite value of K for any value of H as measured on any particular weir, on the assumption of uniform velocity distributions and no friction.

Any experimental determination of the factor, K, differing from the values given would, of course, indicate a departure from the assumptions of Fig. 8 and would give a value of Q from Equation (30), which, if substituted

in Equation (29), would give a value of h_x differing from $\frac{v_x^2}{2g}$ (v_x being the mean velocity, $\frac{Q}{a}$).

From Equations (29) and (30),

$$h_x = k' H \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (36)$$

in which,

$$k' = 0.4717\, K^{\frac{2}{3}} - 1 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (37)$$

also,

$$0.4717 = \frac{3}{2\, g^{\frac{1}{3}}}$$

Coincident values of K and k' from Equation (37) are listed in Table 6, from which it is seen that $k' = 0$ only when $K = 3.087$ (its lower limit), and hence for any greater value of K there must be some velocity of approach in some part of a_x even if a_x is infinitely large. h_x from Equation (36) will equal $\frac{v_x^2}{2g}$ only when K in Equation (37) agrees with Fig. 8.

Line BB of Fig. 8 would indicate values of K from 3.13 to 3.34 for the weir of Fig. 9(b), as H increases from 1 to 4, assuming uniform velocity distribution and no friction, and should be correct if these assumptions were realized. Suppose experiments on this weir give a K of 3.33 for $H = 1$, instead of 3.13, indicating a correction factor applied to Fig. 8 of 1.064 $\left(= \frac{3.33}{3.13}\right)$. Then,

$$Q = 3.33 \times 9.8 \times 1 = 32.634$$

$$\frac{v_x^2}{2g} = \frac{Q^2}{2\, g\, a_x^2} = \frac{(32.634)^2}{2\, g\, (40)^2} = 0.01035$$

and, from Equations (36) and (37),

$$h_x = 0.0518$$

Hence, in this case,

$$h_x = 5\, \frac{v_x^2}{2g}$$

An "effective" velocity head of $5\, \frac{v_x^2}{2\, g}$ would indicate a high degree of non-uniformity in the velocities at Section x, probably a high-surface velocity with perhaps some dead water, or even back-flow, at the invert.

Let H_0 equal the value of H at a point where there is no velocity of approach. Then, $h_x = 0$, and, from Equation (29),

$$Q = 3.087\, b\, H_0^{\frac{3}{2}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (38)$$

from which,

$$H_0 = 0.4717 \left(\frac{Q}{b}\right)^{\frac{2}{3}} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (39)$$

which is the elevation of the reservoir flow line above the crest of a spillway weir, assuming uniform velocity on the crest and no friction losses in approaching the weir. (Q = flow over weir and b = net width of stream on crest.)

Orifices

Since Equation (10) is perfectly general, it would be possible to apply its principles to an orifice. Sections x and y are regarded as being, respectively, above and below the plane of the orifice. At the plane of the orifice, there is an intermediate section, 00. Uniform velocity distributions are assumed at all sections.

$$Q = a_x v_x = a_0 v_0 = a_y v_y \ldots\ldots\ldots\ldots\ldots\ldots (40)$$

Let the head loss between Sections x and y, equal,

$$F = k h_0 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (41)$$

in which, k is a coefficient, depending on local conditions.

$$h_0 = \frac{v_0^2}{2g} = \frac{Q^2}{2 g a_0^2}, \quad h_x = \frac{Q^2}{2 g a_x^2}, \text{ and } h_y = \frac{Q^2}{2 g a_y^2} \ldots\ldots\ldots (42)$$

From Equation (10),

$$(d_x + h_x) = (d_y + h_y) + (k h_0 - I) \ldots\ldots\ldots\ldots (43)$$

S is the water-surface drop between x and y, and $S = d_x + I - d_y$. Hence, from Equation (43),

$$S = k h_0 + h_y - h_x \ldots\ldots\ldots\ldots\ldots\ldots (44)$$

From Equations (44) and (42),

$$2 g S = Q^2 \left(\frac{k}{a_0^2} + \frac{1}{a_y^2} - \frac{1}{a_x^2} \right)$$

and,

$$Q = a_x a_0 a_y \sqrt{\frac{2 g S}{k a_x^2 a_y^2 + a_0^2 a_x^2 - a_0^2 a_y^2}} \ldots\ldots\ldots\ldots (45)$$

Compare Equation (45) with the following formula:

$$Q = c a_0 \sqrt{2 g S} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (46)$$

in which, c is a "coefficient of discharge" in terms of S.

From Equations (45) and (46),

$$k = \left(\frac{1}{c}\right)^2 - \left(\frac{a_0}{a_y}\right)^2 + \left(\frac{a_0}{a_x}\right)^2 \ldots\ldots\ldots\ldots\ldots (47)$$

$a_y = c_1 a_0$, in which, c_1 is the coefficient of contraction, and, hence,

$$k = \frac{1}{c^2} - \frac{1}{c_1^2} + \left(\frac{a_0}{a_x}\right)^2 \ldots\ldots\ldots\ldots\ldots\ldots (48)$$

Equations (47) and (48) lead to the following special formulas:

If

$$a_x = a_y,$$

$$k = \frac{1}{c^2} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (49)$$

If the velocity of approach is zero, $a_x = \infty$, and,

$$k = \frac{1}{c^2} - \frac{1}{c_1^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (50)$$

If c_1 in Equation (50) is 1.0,

$$k = \frac{1}{c^2} - 1 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (51)$$

If c_1 in Equation (50) is 0.6,

$$k = \frac{1}{c^2} - 2.78 \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (52)$$

TABLE 8.—COINCIDENT VALUES OF c AND k.

Value of c.	Values of k.		
	From Equation (49).	From Equation (51).	From Equation (52).
0.5	4.0	3.0	1.22
0.6	2.78	1.78	0
0.8	1.56	0.56	....
1.0	1.0	0	....

It is to be remembered that c and k in Table 8 are coefficients, such that, $Q = c\, a_0 \sqrt{2\, g\, S}$ (Equation (46)) and $F = k\, h_0$ (Equation (41)).

Since F cannot be negative, k cannot be negative and, hence, from Equation (50), it follows that, in the case of zero velocity of approach, c cannot be greater than c_1.

From Equations (42) and (46),

$$h_o = c^2\, S \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (53)$$

and, hence, from Equation (41),

$$F = k\, c^2\, S \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (54)$$

and,

$$k\, c^2 = \frac{F}{S} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (55)$$

It may be that this discussion of orifices is more academic than useful, but it brings out some interesting relations and demonstrates the general applicability of the basic equations.

SIPHONS

In the basic equations, (6), (10), and (11), d is the vertical distance from the invert to the "hydraulic gradient". The "hydraulic gradient" (Line B, Fig. 1) is the water surface profile on the assumption of equal barometric pressures on the water surfaces at all sections.

Let D = the water head corresponding to the barometric pressure; and d' = the vertical distance from the invert to the water surface.

Then the basic energy equation, Equation (6), may also be expressed as:

$$d'_x + D_x + h_x + I = d'_y + D_y + h_y + F \quad \ldots\ldots\ldots\ldots\ldots (56)$$

If D is constant, the water surface profile and the "hydraulic gradient" coincide, and d and d' are synonymous at all sections.

Subtracting Equation (6) from Equation (56),

$$d'_x + D_x - d_x = d'_y + D_y - d_y \quad \ldots\ldots(57)$$

Suppose the water surface coincides with the hydraulic gradient at Section x. Then, $d_x = d'_x$ and from Equation (57),

$$d'_y = d_y + (D_x - D_y) \quad \ldots\ldots(58)$$

On the further supposition that $D_y = 0$

$$d'_y = d_y + D_x \quad \ldots\ldots(59)$$

Equation (59) represents the conditions existing when Section x is in an open channel or pool and Section y is in a closed pipe running full. In this case, D_x is the atmospheric head and d'_y is the vertical distance from the invert to the water level in a vertical vacuum tube at Section y and is the maximum possible water depth at that section.

The theoretical value of D_x is about 34 ft., but as a matter of practice it is seldom safe to assume D_x as greater than 28 ft.

In computing a full conduit or siphon down stream from open head-water, the first application of Equation (56) would be as follows:

$$(d'_y + h_y) = (d'_x + D_x + h_x) + (I - F) \quad \ldots\ldots(60)$$

In the next succeeding stretch of conduit, Section y becomes Section x, D_x becomes zero, and,

$$(d'_y + h_y) = (d'_x + h_x) + (I - F) \quad \ldots\ldots(61)$$

Since h and F are independent of d in a full conduit, Equation (61) will produce the same results as Equation (11) unless d, at some section, becomes less than the vertical depth of the conduit at that point. In that case the conduit, instead of flowing at the depth, d, will continue to flow full unless d' (= d + atmospheric head) becomes less than the full conduit depth.

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PAPERS AND DISCUSSIONS

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FOUNDATIONS AND DRAINAGE OF HIGHWAYS*

By Albert C. Rose,† Assoc. M. Am. Soc. C. E.

Synopsis

This paper is based on field studies made by the writer in the Pacific Northwest during 1920 and 1921. The object of the studies was the development of simple and practical field tests and methods of identifying good, fair, and bad sub-grade soils.

As a result of the investigation the volumetric shrinkage of the soil was concluded to be the fundamental adverse factor. For measuring this, a linear shrinkage test was devised, as the displacement of pavements seemed to be caused principally by the distortion of the sub-grade in one direction—vertically. The procedure for the field moisture equivalent test was devised to prepare soils for the linear shrinkage test by wetting them with comparable quantities of water; but later it came to have a value of its own.

These tests were used on existing sub-grade soils, and the corresponding pavement conditions were observed. It was noted that an increase in the clay content of the soil, which was indicated by the test values, was accompanied by a corresponding adverse condition of the pavement.

Taken together, the field observations led to the conclusion that certain test limits could be used to identify good, fair, and bad sub-grade soils for the purpose of making a detailed sub-grade survey. A trilinear soil-classification chart, used in combination with a U. S. Bureau of Soils map, further simplified the work and made possible reconnaissance sub-grade surveys without detailed tests.

The linear shrinkage and field moisture equivalent percentages were used to determine the character of the sub-grade soil as a foundation. The stability ratio was used to measure the drainage conditions.

Introduction

The proper design and construction of highway foundations and the methods of providing adequate drainage are research problems toward the

Note.—Written discussion on this paper will be closed in May, 1928.

* Presented at the Spring Meeting, Asheville, N. C., April 22, 1927.

† Assoc. Highway Engr., Bureau of Public Roads, U. S. Dept. of Agriculture, Washington, D. C.

solution of which the up-to-date highway engineer is devoting an intensive effort. The isolation of the sub-grade variable within recent years has accelerated the study of this fundamental road problem—the foundation. Intimately associated with the study of the character of the foundation is the kind of surface that is appropriate for a particular location.

The study of highway foundations and of drainage in connection with the surfaces involves two phases: (1) The economic; and (2), the engineering. The first determines the broad general policies, which establish the location of the road between controlling points; the amount of public revenues available for the work; the existing traffic requirements; a forecast of future development; the cost of operation of vehicles over various types of surfaces; and kindred subjects. The second, the engineering phase, although no less important, is more limited in its scope. It prescribes the design of grades and alignment to suit the terrain; measures the drainage requirements; plans the cross-section and surface of the road; and takes care of all the work involved in the design, construction, and maintenance of the roadway.

Only the engineering features of the problem will be discussed in this paper, and they will be limited principally to showing methods of detecting good and bad sub-grades and drainage, and to outlining the problem of designing surfaces. A careful analysis of engineering literature indicates that pavement, or surfacing, design involves four major variables: (1) The character of materials in, and the quality of workmanship on, the surfacing; (2) the local climatic conditions; (3) the character and condition of the sub-grade soil; and (4) the kind and amount of traffic. Probably at some future time these variables may be evaluated within broad limits, so that the highway engineer may be able to determine a rational surfacing design for any section of road when the local values of the variables are known. The belief that the evaluation may be accomplished in the near future is strengthened by the attention now being devoted to the problem by the various State highway departments, the Highway Research Board, and other research agencies. The U. S. Bureau of Public Roads has initiated recently a nation-wide program for the study of sub-grade soils.

Of the four variables mentioned, the sub-grade seems to be the most important; and in the sub-grade the volume changes caused by variations in moisture content and frost action are the factors that seem to be of greatest concern to the highway engineer. It is granted that no road surface would go to pieces without heavy traffic, but the distortion of the sub-grade due to volumetric changes must first take away the support from the surfacing before a pavement will go to pieces. As far as the other two variables are concerned, either they may be controlled, as by proper workmanship or material, or allowance may be made in a design for the known climatic conditions.

Vertical Deformation the Fundamental Adverse Characteristic of Sub-Grade Soils

Analysis of the California Study.—In a recent paper,* C. A. Hogentogler, Assoc. M. Am. Soc. C. E., presents an enlightening and constructive analysis

* "California Road Survey Demonstrates the Economic Possibilities of Subgrade Studies," by C. A. Hogentogler, *Public Roads*, Vol. 7, No. 12, February, 1927.

of the relative effect of various types of sub-grade soils on pavement conditions covering about 1 200 miles of concrete surface in the California State Highway System in 1920. His tabulated analysis (Table 1) indicates that the undesirable conditions of the sub-grade soils increase with the clay content. On this basis, soils may be classified with regard to their influence on pavement condition.

TABLE 1.—PAVEMENTS CLASSED AS DESIRABLE OR UNDESIRABLE, USING ONLY THREE SUB-GRADE CLASSIFICATIONS.

Type of sub-grade.	PAVEMENT CONDITION.	
	Desirable percentage.	Undesirable percentage.
Sand	100.0	....
Loam	93.0	7.0
Clay	79.6	20.4

These same general conclusions were advanced by the writer after making a number of field studies of sub-grade soils as to their effect on pavements in the States of Oregon and Washington during 1920 and 1921. These investigations were prosecuted with equal diligence at all seasons of the year so that the results represent the extremes as well as average conditions of the sub-grade soils as affected by climatic variations.

Development of Field Moisture Equivalent and Linear Shrinkage Tests.—At the time these field studies were initiated the writer discussed the problem with highway engineers and agricultural soil experts, consulted soil bibliographies and literature, and made reconnaissance surveys of existing field conditions. Based on this preliminary information the indications were that the volumetric change in the sub-grade soils was the fundamental factor causing cracking, displacement, and other unfavorable conditions of pavements. This volumetric change might be caused either by variations in the moisture content of the soil or by frost action. It was evident, therefore, that some test should be devised for measuring the comparable volumetric changes between soils varying in the size, as well as in the character, of the component grains. It was further evident that this test should measure the linear shrinkage rather than the volumetric shrinkage of the soil since the distortion of the sub-grade that caused pavement displacement acted mainly in one direction—vertically.

The standard moisture equivalent test seemed to be the most promising method for wetting the soils with comparable quantities of water prior to making shrinkage tests.* It was considered probable that, in general, a force 1 000 times that of gravity would leave a surface film of water of equal thickness around all soil grains even if they vary in size and character. This would

* The making of shrinkage tests was suggested by C. L. McKesson, Assoc. M. Am. Soc. C. E., now Materials and Research Engineer of the California State Highway Commission. The writer carried on the field studies for the Bureau under Mr. McKesson's immediate direction.

depend, of course, on whether the individual grains were porous or solid, or had a glazed or roughened surface.

An examination of the standard centrifuge for making the moisture equivalent test showed it to be a machine designed for the laboratory and unsuitable for the general field use of the highway engineer. It was decided, therefore, to duplicate if possible, with an inexpensive but simple and practical field test, the results secured with the standard apparatus for determining the moisture equivalent.

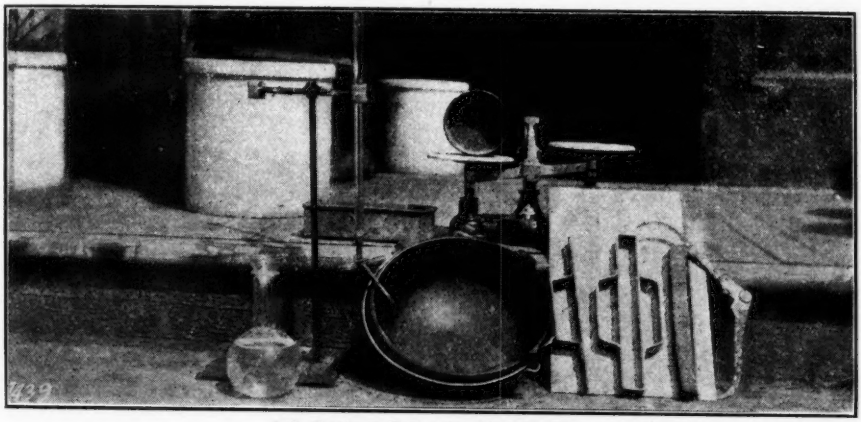
The quantity of water absorbed by a soil due to surface tension and adhesion was compared with the water content retained by the soil against a force equivalent to 1 000 times that of gravity (the standard centrifuge force). To do this the soil was powdered to pass a 1-mm. screen and water was dropped from a burette on the sample in an iron bowl with constant stirring. When the material attained the consistency of putty, it was compacted and the water was allowed to drop on the surface as long as it was absorbed. When the absorption was complete the surface assumed a wet, shiny appearance, which indicated the critical point of the test. Fig. 1 illustrates three stages in the test.

The water content of the soil divided by the dry weight of the soil and the result multiplied by 100 gave the "field moisture equivalent". It was found that this percentage could be determined accurately for soils if it were 20, or greater. Further field investigation showed that where soils gave lower values they were so coarse-grained as to drain readily, and, therefore, were practically always good sub-grade soils. Consequently, extreme accuracy for soils with a field moisture equivalent of less than 20 was not considered of practical significance.

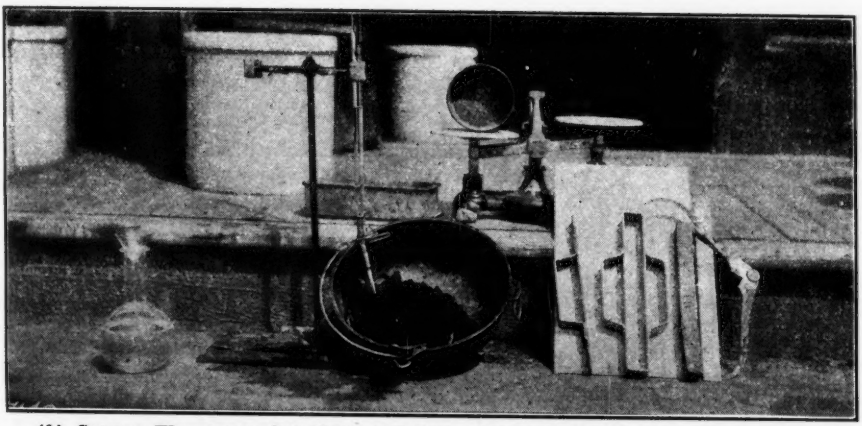
Comparison of Field Moisture Equivalent with Standard Moisture Equivalent.—An analysis by least squares of the field moisture equivalents for twenty-nine soil samples, with values ranging from 20 to 41%, indicated that they were slightly closer to the standard moisture equivalent results, on portions of the same samples centrifuged in the laboratory of the U. S. Bureau of Soils, than similar laboratory tests made by the Oregon Agricultural College.* The field moisture equivalent method was used, therefore, for a comparative wetting of various soil samples before they were moulded into 1 by 1 by 10-in. bars (Fig. 2) and shrunk to determine the linear shrinkage value. This was considered to be the difference between the wet and dry length of the bars computed as a percentage of the wet length.

Check runs of these two simple and practical field tests indicated that, while they did not give identical results they gave consistent values, as close as those obtained by other routine engineering tests. It was found that a general relation existed between the values of the field moisture equivalent and the linear shrinkage, and that for tentative investigations the simpler test (field moisture equivalent) was indicative of the results to be obtained by the more difficult test (linear shrinkage). For absolutely accurate work, however, the linear shrinkage was considered the critical test.

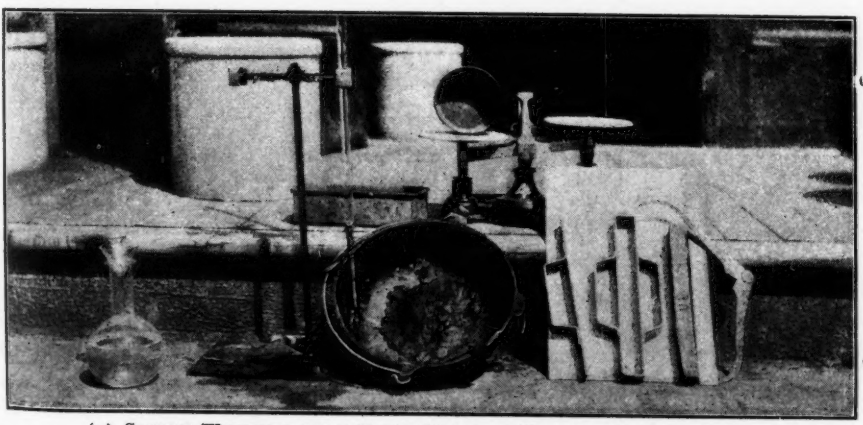
* "Practical Field Tests for Subgrade Soils," by A. C. Rose, *Public Roads*, Vol. 5, No. 6, August, 1924.



(*a*) Sample Dry and Pulverized.

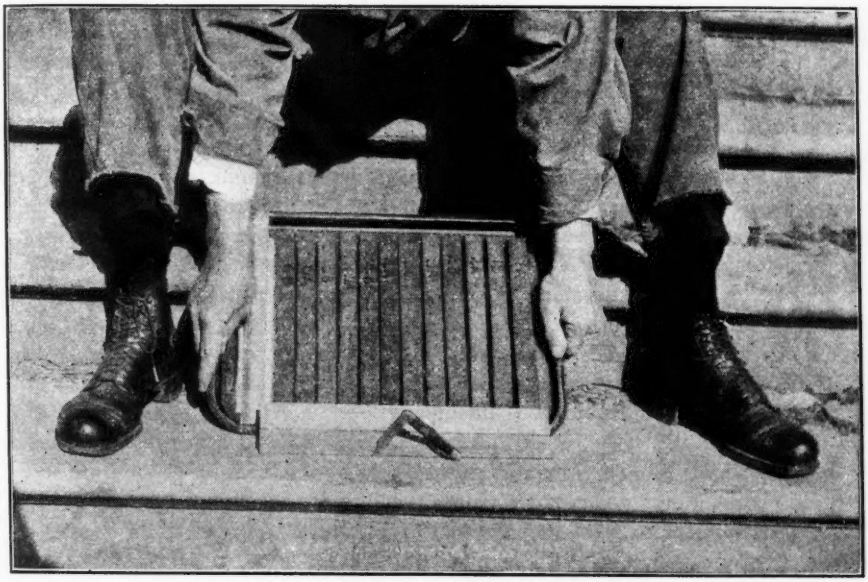


(*b*) Sample Wetted to One-Half the Field Moisture Equivalent Percentage.



(*c*) Sample Wetted to the Total Moisture Equivalent Percentage.

Fig. 1.—Apparatus for Making Field Moisture Equivalent Test.



(*a*) Soil Bars Before Being Shrunk.

(*b*) Soil Bars After Being Shrunk.

Fig. 2.—Determination of Linear Shrinkage.

Later, it was observed that the field moisture equivalents of a large number of soils were numerically similar to the clay contents of the soil in percentage as determined by the mechanical analyses of the U. S. Bureau of Soils shown in its *Soil Survey Bulletins.*

Comparison of Pavement Conditions with Test Values

Subsequent to the development of these tests the writer made a number of field studies to determine what, if any, relation existed between the condition of the pavement and the test values of the sub-grade soil. Out of these investigations came the conclusion that the sub-grade soil was more important in its adverse effect on the pavement than the kind or amount of traffic. Pavements may crack as a result of temperature variations, but they are rarely broken up and disintegrated by traffic, except when on an unstable sub-grade or on one distorted by unequal volume changes in the sub-grade soil. Under the latter condition rigid pavements act as cantilevers or beams and are more readily broken, while non-rigid pavements become depressed, shattered, or broken through.

Sub-Grade Failure Under a Concrete Pavement.—One of the investigations covered the sub-grade failure under a concrete pavement, 2 miles long, between Centralia and Chehalis, Wash., laid during June, July, and August, 1914 and 1915. The severity of a part of the failure is illustrated by Figs. 3 and 4. The adverse humid climate of the region is shown by Fig. 5. During construction the average temperature was 65° Fahr. Since then the variation has been from a maximum of 99° Fahr. to a minimum of —16° Fahr., with a maximum contraction range of 81° Fahr. and a maximum expansion range of 34° Fahr.

The pavement was divided into four sections based on the degree of failure in each portion. In each of these sections a 400-ft. length was selected as representative. The detailed tests were confined to these 400-ft. sections.

TABLE 2.—Summary of Representative Soil Test Values on Tests Nos. 1 to 4, Inclusive.

Test number.	Section of pavement represented, station to station.	Relative condition of pavement in section.	California study classification.	Classification of soil by U. S. Bureau of Soils.	Soil Test Values.	
					Percentage of field moisture equivalent.	Percentage of linear shrinkage.
1	0 to 56	Best.....	*C*	Chehalis clay loam.	30.8	2.1
2	56 to 65	Second best..	*D*	Chehalis clay loam.	35.6	6.6
3	93 to 107 + 84	Third best....	*D*	Transition between Chehalis clay and clay loam.....	42.4	11.3
4	65 to 93	Worst.....	*E*	Chehalis clay.....	55.1	13.1

The investigation showed unmistakably that the degree of failure of the test sections corresponded with the increase in the values of the field moisture equivalents and linear shrinkages of the sub-grade soil. The latter are summarized in Table 2. The corresponding condition of the test sections is repre-

sented by crack charts (Fig. 6) and photographs (Figs. 7 to 10, inclusive). An adjacent section of concrete pavement laid on a sandy soil, with a linear shrinkage percentage of zero, was practically free from cracks or other defects.

The failure of this pavement could not be attributed to defective concrete. Although tensile and compressive tests were not made, it was evident that the concrete was good. It was sound under the blow of a hammer and when broken showed a normal fracture. The construction inspector's records indicated that an adequate quantity of cement had been used for the specified mixture.

The increase in the clay content of the soil corresponded with the degree of severity of the failure even where the sub-grade varied from cut to fill, and where the fill varied in height and was overflowed annually by the flood water of the adjacent river. The mechanical analyses of the soil types, as made by the U. S. Bureau of Soils, are shown in Table 3.

TABLE 3.—THE MECHANICAL ANALYSES OF SOIL TYPES ENCOUNTERED ON TESTS NOS. 1, 2, 3, AND 4, AS DETERMINED BY THE U. S. BUREAU OF SOILS.

Type.	Description.	Percentage of fine gravel.	Percentage of coarse sand.	Percentage of medium sand.	Percentage of fine sand.	Percentage of very fine sand.	Percentage of silt.	Percentage of clay.
Chehalis silty clay loam.....	Soil.....	0.0	0.1	0.1	2.6	5.8	64.8	26.4
	Subsoil...	0.0	0.0	0.2	1.9	1.0	64.7	32.1
Chehalis clay loam	Soil.....	None listed.						
	Subsoil...	None listed.						
Chehalis clay.....	Soil.....	0.0	0.1	0.1	0.7	0.2	24.0	74.6
	Subsoil...	0.0	0.0	0.0	0.3	0.5	20.7	78.3

Sub-Grade Failure Under a Bituminous Pavement.—Another serious sub-grade failure was investigated under a Warrenite bituminous pavement, 4 852 ft. long, extending north from the southern city limits of Chehalis, Wash.

The pavement failures were classified as: (1) Depressed areas (D. A.); (2) incipient failures (I. F.); (3) shattered areas (S. A.); and (4) broken through areas (B. T.). Any depression that averaged more than 0.05 ft. below a 6-ft. straight-edge was considered a depressed area. Incipient failures were depressed areas or others in which the pavement had begun to fracture but no shattering or disintegration had yet occurred. The areas were measured until the observer became sufficiently familiar with the dimensions to estimate them. An automobile speedometer was used to locate the failures along the project. A sample of the form used is shown in Table 4.

The tests of the sub-grade soil showed the field moisture equivalents to vary from 26 to 45 per cent. From previous tests,* the corresponding linear shrinkages for these moisture equivalents were estimated at 3 to 9 per cent. The soil map made by the U. S. Bureau of Soils over this area showed the predominating soil types to be members of the Salkum series. The mechanical analyses in Table 5 show the clay content to be well in excess of 30 per cent.

* "Practical Field Tests for Subgrade Soils", by A. C. Rose, *Public Roads*, Vol. 5, No. 6, August, 1924.

FIG. 3.—SEVERE CRACKING OF CONCRETE PAVEMENT ON CENTRALIA-CHEHALIS ROAD, WASHINGTON.

FIG. 4.—TYPE OF CONCRETE PAVEMENT FAILURES REPRESENTED BY TEST 4, CENTRALIA-CHEHALIS ROAD, WASHINGTON.

The investigation indicated that the 2-in. Warrenite wearing surface, on a 2-in. new stone course, over a 6-in. old macadam road, was not heavy enough to withstand the traffic under the existing adverse soil conditions and the inadequate surface drainage and sub-drainage. As a whole, the pavement was a failure, 34.6% of the total area being defective, distributed throughout the entire length of the pavement.

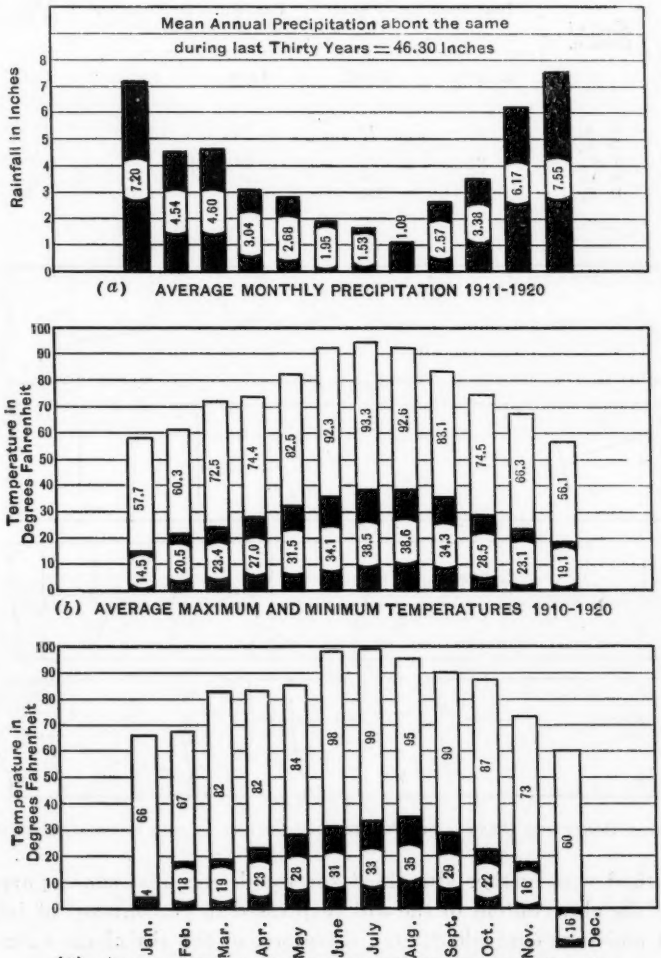


FIG. 5.—CLIMATOLOGICAL DATA FOR REGION IN VICINITY OF CENTRALIA AND CHEHALIS, WASH., COMPILED FROM RECORDS OF U. S. WEATHER BUREAU.

Results of the Field Investigations in the Pacific Northwest.—The foregoing two examples illustrate the findings of the sub-grade studies in the Pacific Northwest. As a result the linear shrinkage of the soil was considered to be the critical test and the field moisture equivalent was taken as a simple

and rapid indication of this value. The exceptions to the rule have been noted in previous papers by the writer.*

TABLE 4.—SAMPLE OF FORM USED FOR CLASSIFYING PAVEMENT DEFECTS IN BITUMINOUS PAVEMENT REPRESENTED BY TESTS NOS. 5 AND 6.

Mile.	Kind of failure.	Dimensions, in Feet.		Distance from Edge of Pavement to Nearest Point of Failure, in Feet.		Area Effected, in Square Feet.	
		Length.	Width.	Left.	Right.	Left.	Right.
0.30	D. A.	3	10	5	..	30	.....
0.48	D. A.	5	18	..	0	.....	90
0.58	I. F.	2	2	7	..	4	.....
0.70	S. A.	5	6	..	6	.....	30
0.75	B. T.	18	150	0	0	1 350	1 350

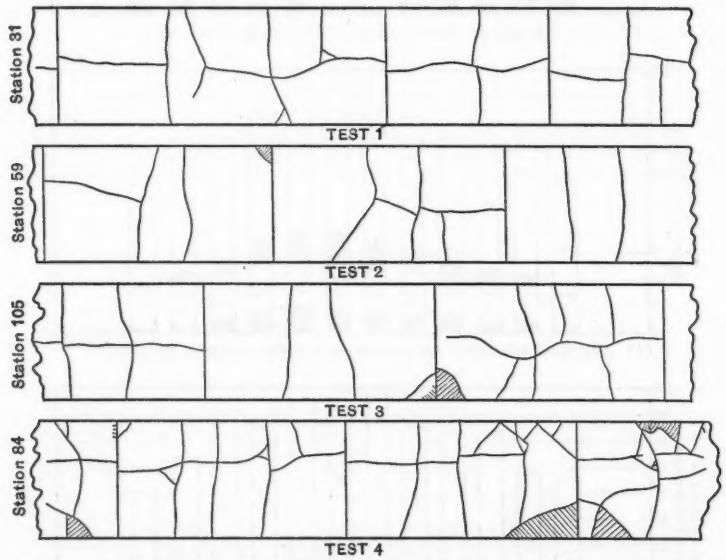


FIG. 6.—TYPICAL CONDITION CHARTS OF CONCRETE PAVEMENT AT THE SEVERAL TEST SECTIONS.

The method was further simplified for rough reconnaissance purposes by considering the clay content of the soil (expressed in percentage) as indicative of the field moisture equivalent, and, therefore, of the shrinkage value of the soil, a relation that experience had shown to be reasonably safe. The advantage to be gained by using this relation lies in the fact that it enables the highway engineer to interpret, for purposes of sub-grade study, the great volume of data already made available by the U. S. Bureau of Soils, showing the mechanical analyses of soils of all varieties found in all sections of the

* "Practical Field Tests for Subgrade Soils", *Public Roads*, Vol. 5, No. 6, August, 1924; "Field Methods Used for Making Subgrade Surveys," *Public Roads*, Vol. 6, No. 5, July, 1925; and "The Present Status of Subgrade Studies," *Public Roads*, Vol. 6, No. 7, September, 1925.

FIG. 7.—VIEW OF PAVEMENT IN TEST 1.

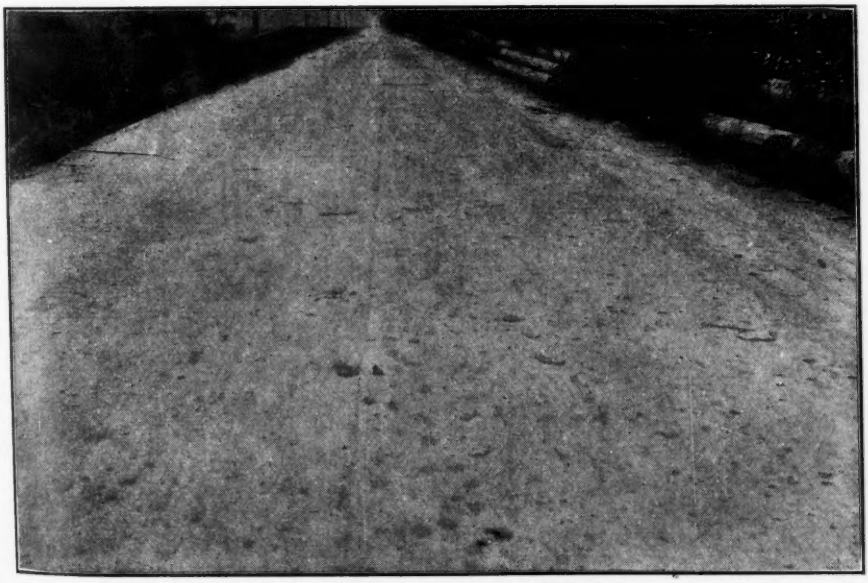


FIG. 8.—SECTION OF TEST 2 PAVEMENT.

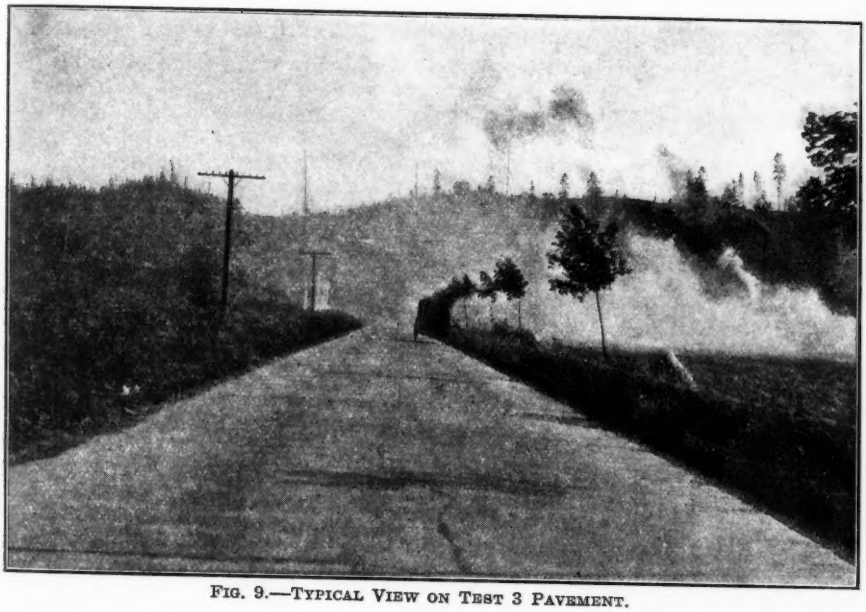


FIG. 9.—TYPICAL VIEW ON TEST 3 PAVEMENT.

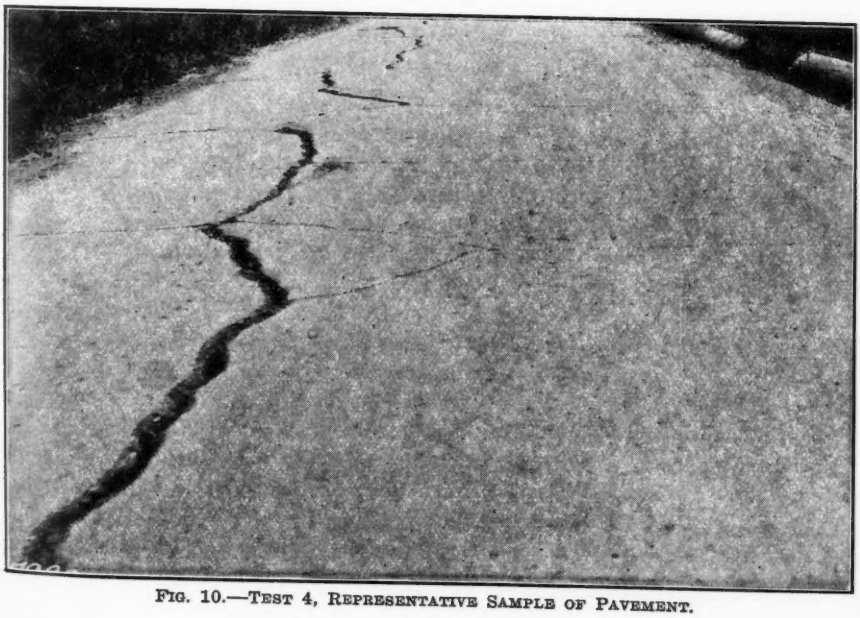


FIG. 10.—TEST 4, REPRESENTATIVE SAMPLE OF PAVEMENT.

country. For purposes of ready interpretation of these data a trilinear soil classification chart was developed (Fig. 11). The classification of sub-grade soils by texture is, as follows:

Soils containing less than 20% of clay:

Coarse sand—more than 25% of fine gravel and coarse sand, and less than 50% of any other grade.
Sand—more than 25% of fine gravel, coarse and medium sand, and less than 50% of fine sand.
Fine sand—more than 25% of fine sand or less than 25% of fine gravel, coarse and medium sand.
Very fine sand—more than 50% of very fine sand.

Soils containing 20 to 50% of silt and clay:

Sandy loam—more than 25% of fine gravel, coarse and medium sand.
Fine sandy loam—more than 50% of fine sand or less than 25% of fine gravel, coarse and medium sand.
Very fine sandy loam—more than 35% of very fine sand.

TABLE 5.—MECHANICAL ANALYSIS OF SALKUM SILTY CLAY AS DETERMINED BY THE U. S. BUREAU OF SOILS.

Type.	Description.	Percentage of fine gravel.	Percentage of coarse gravel.	Percentage of medium sand.	Percentage of fine sand.	Percentage of very fine sand.	Percentage of silt.	Percentage of clay.
Salkum silty clay	Soil.....	0.5	1.2	1.1	3.3	3.1	59.4	31.5
	Subsoil.....	0.2	1.1	1.2	3.2	3.6	55.6	34.6

The type names in the several areas of Fig. 11 agree with the nomenclature used by the U. S. Bureau of Soils in making its soil maps.

METHOD FOR MAKING SUB-GRADE SURVEYS DEVELOPED FROM FIELD STUDIES

Correlation of Test Values with Existing Soil Maps.—With this trilinear soil classification chart, the problem of making a reconnaissance sub-grade soil survey in areas mapped by the U. S. Bureau of Soils is greatly simplified. The maps show the location of the various soil types and the points on the roads at which the character of the soil changes. The *Soil Survey Bulletins* published by the Bureau contain mechanical analyses of all the soils shown on the maps. The nomenclature of the maps and the *Bulletins* agree with that of the trilinear soil classification chart.

It becomes a simple matter then for the engineer studying sub-grade conditions to determine from the maps the type of the soil at particular locations, and, by the use of the *Bulletins* and the trilinear chart and by the employment of the observed relation between the field moisture equivalent and clay content, to distinguish between the soils that are clearly good sub-grades and those that are clearly bad sub-grades. Of course, there will always be some soils of doubtful character, which it will be necessary to test either by the simplified methods described herein or by more elaborate laboratory methods; but the number of such cases is relatively small.

The field observations indicated that for the purpose of making sub-grade soil reconnaissance surveys, the clay content by mechanical analysis may be considered equal to the field moisture equivalent. When the field moisture equivalent was less than 20, the linear shrinkage percentage was usually negligible and the soil made a good sub-grade; when the field moisture equivalent varied from 20 to 30, the linear shrinkage percentage usually did not exceed 5 and the soil made a fair sub-grade; and when the field moisture equivalent exceeded 30, the linear shrinkage percentage usually was greater than 5% and the soil made a poor sub-grade.

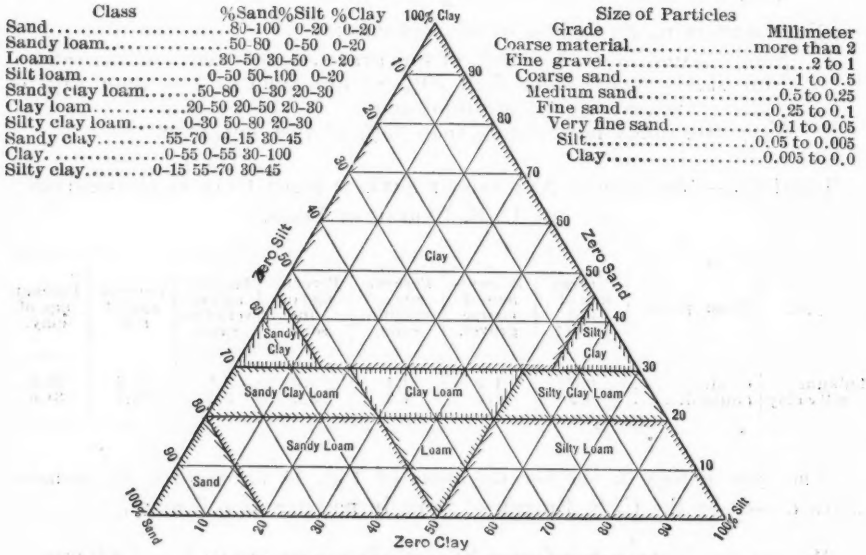


FIG. 11.—TRILINEAR SOIL CLASSIFICATION CHART.

The results vary considerably in individual cases and are based on observations made in the Pacific Northwest. Before they are accepted for any other region, or for any particular project, they should be verified by actual tests.

The tests for field moisture equivalent and linear shrinkage are not claimed to indicate good, doubtful, and bad sub-grade soils under all conditions. No tests, however refined, that have yet been devised, will accomplish this. In the field studies in the Pacific Northwest certain pavement conditions could not be explained on the basis of the field test values. Although not 100% effective, the tests were, however, significant, in the great majority of cases, and it is believed that they will be of value to the field engineer until some simpler and more effective method is devised.

They were also found to be usable by a highway engineer with only moderate preliminary training. They are now being used by the State of California and have been included in those under investigation by the U. S. Bureau of Public Roads. Frank H. Eno, M. Am. Soc. C. E., in his summary of the

progress of soil investigations states* that "Rose's limits for shrinkage at 5 per cent. and moisture equivalent at 20 per cent. seem to be working out satisfactorily in practice where they have been used."

The writer believes that the results secured with the field tests developed in the Pacific Northwest and their subsequent use by other States have demonstrated their usefulness for the detection of good, doubtful, and bad sub-grade soils. The consistency of the test results has been verified in the laboratory of the U. S. Bureau of Public Roads by the close agreement between check tests made by the same investigators and between those on the same samples made by different investigators. They are advocated primarily as field tests designed for simplicity and practicability. They give a direct determination of the fundamental adverse factor of the sub-grade soil—the volume change in one direction (the vertical), represented by the linear shrinkage percentage. In addition, test limits have been devised based on the values obtained by these tests, which are useful criteria for determining the relative character of sub-grade soils.

Sub-Grade Soil Classification Now in Progress.—The Bureau of Public Roads with the aid of its laboratory has recently initiated an elaborate investigation to determine the upper and lower test limits of the various soil series and types throughout the country, as identified by the nomenclature of the Bureau of Soils. This study extends over all the States—investigations that previously had been confined to individual States scattered throughout the West and East. These soil tests will be carried on in connection with studies of pavements in the various regions, the object being to correlate definite soil test values with the existing pavement conditions. In order to reduce the required laboratory testing to a minimum it is planned to group all sub-grade soils tentatively on the basis of the values secured by the Atterberg plasticity test,† which is comparatively simple to perform and gives information on soil characteristics varying from low to high values.

Later, more elaborate tests will be made on representative samples of each of these groups, to determine the maximum and minimum values of such engineering characteristics as the compressibility, elasticity, and permeability. The values will be determined for each soil horizon (or depth below the surface) for each group, so that the highway engineer may know the effect of raising or lowering the grade line. By this method, in connection with correlated observations on existing road conditions, it is hoped to identify all the U. S. Bureau of Soils types and series with regard to their efficiency as sub-grade soils. Under this plan Melbourne clay might be found to fall within the doubtful group, while a Chehalis clay might always lie in the bad group. Also, it might be found best to lay the grade line of the highway in the good surface horizon rather than on the bad sub-soil.

When this work is completed it is hoped that the soil series and types as mapped by the U. S. Bureau of Soils will be converted into terms that are

* "Progress in Subgrade Soil Investigations," by Frank H. Eno, Paper delivered before the Twenty-fourth Annual Convention of the Am. Roadbuilders Assoc., Chicago, Ill., January 13, 1927.

† "Simplified Soil Tests for Subgrades and Their Physical Significance," by Charles Terzaghi, M. Am. Soc. C. E., Mass. Inst. Tech., *Public Roads*, Vol. 7, No. 8, October, 1926.

usable for the highway engineer. The objective of all these laboratory tests and correlated investigations on road conditions will be to reduce the necessary testing to a minimum. Thus, for any given region, the soils will be classed with respect to their physical characteristics, and later, when field survey data have been received, they will be classed with regard to the physical condition of the pavements laid on them.

Until the results of these tests become available, however, and even after the work is virtually completed, there will always be a need for practical and inexpensive field tests to identify the character of the doubtful soils. It is to satisfy this need that the field tests and methods suggested in this paper have been formulated.

Definition of the Stability Ratio

In making the shrinkage tests it was noticed that the soil could not be compacted into solid bars when the moisture content exceeded the field moisture equivalent. This same plasticity was observed in making the field auger borings. When the moisture content was less than the field moisture equivalent, the drillings issued from the earth in hard, shiny "worms". When the moisture content was excessive, the condition of the soil varied from sloppy to putty-like, to soft, to friable; and the drill could be pushed down without rotation.

For these reasons the stability ratio of the soil was defined as the moisture content divided by the field moisture equivalent (both in percentage).* When the stability ratio of a sub-grade soil is greater than 1, the soil is considered to be in an unfavorable condition. This ratio applies only to the fine-grained soils with a field moisture equivalent in excess of 20. The coarse-grained soils drain readily and do not seem to be materially affected by a like moisture content.

The observations on well-drained soils in the Pacific Northwest in the open fields, or in highway rights of way not under a pavement surface, indicated that the maximum moisture contents of the soils were less than the field moisture equivalents. This applied to samples taken far enough beneath the surface (about 3 to 6 in.) to eliminate the effect of water standing in slight depressions or of sheets of run-off water. This condition was found to exist even in the rainy season and within 3 or 4 hours after heavy rainfalls.

On poorly drained soils, where surface depressions or restricted sub-drainage existed, the moisture content of the natural soils, at depths below the surface comparable with the well-drained soils, generally exceeded the field moisture equivalent.

In 1925, Professor F. J. Alway, of the University of Minnesota, stated that sufficient investigations had been made over the country at large to show that in well-drained soils the maximum moisture contents throughout the year usually were equal to or less than the moisture equivalent. It should be noted, however, that Professor Alway's statement referred to the standard moisture equivalent made against a force equivalent to 1 000 times that of gravity.

* "Practical Field Tests for Subgrade Soils," by A. C. Rose, *Public Roads*, Vol. 5, No. 6, August, 1924.

Although the field moisture equivalent seems to approximate the standard moisture equivalent closely, it is not made against any force except the surface tension of the water about, and the adhesion to, the soil particles. Therefore, it should not be confused with the standard laboratory test.

On observations under pavements, with few exceptions, the moisture contents of the soil at points of failure exceeded the field moisture equivalent. The converse was true for sections of pavement in good condition.

The field observations on natural soils not covered by pavements, as well as on sub-grade soils, indicate that there is a possibility by proper pavement design of controlling the moisture content in a sub-grade soil so that it will rarely exceed the field moisture equivalent.

Summary

These studies of the engineering factors in highway foundation and drainage show definite prospects of ultimate benefit. In the near future the variables may be evaluated within broad limits, so that it will be possible to determine an adequate pavement design for any section of road where these factors are known. The following conclusions may be drawn:

1.—The adverse character of the sub-grade soil depends primarily on the volume change caused by variations in moisture content or frost action. The main adverse action of the volume change occurs in one direction—the vertical.

2.—The volume changes of the soil distort the sub-grade, which displaces the pavement unevenly and subjects it to excessive localized stresses.

3.—The adverse character of the sub-grade soil is shown to increase with the clay content.

4.—The clay content and adverse character of the sub-grade soil is indicated by certain simple and practical field tests, called the field moisture equivalent and the linear shrinkage tests.

5.—Detailed sub-grade soil surveys may be readily made by using U. S. Bureau of Soils maps in connection with these field tests.

6.—A simplified and rapid method for making a reconnaissance soil survey is by the use of a U. S. Bureau of Soils map in connection with a trilinear soil classification chart, without the use of field or other tests. Generally speaking, sands, sandy loams, loams, and silty loams are considered good sub-grade soils; sandy clay loams, clay loams, silty clay loams, sandy clays, and silty clays, doubtful sub-grade soils; and clays, bad sub-grade soils.

7.—The drainage condition of a sub-grade soil, as well as the bearing power, is indicated as poor when the critical point—a stability ratio of one—is exceeded. The stability ratio is defined as the moisture content divided by the field moisture equivalent.

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DESIGN AND CONSTRUCTION OF CONCRETE PAVEMENTS*

By Clifford Older,† M. Am. Soc. C. E.

Much pertinent information has been published during the past several years, concerning the design, construction, and advantages of concrete pavements. While the data available cover the field quite thoroughly, perhaps an emphasis of some of them may be justified.

Rational Computation of Stresses

In the field of design it would seem that the most outstanding improvement has been contributed by H. M. Westergaard, M. Am. Soc. C. E., in his paper entitled "Stresses in Concrete Pavements Computed by Theoretical Analysis".‡ Inasmuch as his equations take into account all essential factors, including the effect of sub-grade support, they provide a means for predetermining the stresses that may be expected under any given set of conditions.

Formulas.—Professor Westergaard derives the following equations:

For maximum stress due to a single wheel load at a corner:

$$S_c = \frac{3\,P}{h^2}\left[1 - \left(\frac{E\,h^3}{12\,(1-u^2)\,k}\right)^{-0.15}(a_1{}^{0.6})\right] \quad \ldots\ldots (1)$$

For maximum stress at an edge some distance from a corner:

$$S_e = 0.572\,\frac{P}{h^2}\left[\log_{10}(h^3) - 4\log_{10}\left(\sqrt{1.6\,a^2 + h^2} - 0.675\,h\right) - \log_{10} k + 5.767\right] \quad \ldots\ldots (2)$$

For maximum stress at interior points:

$$S_i = 0.3162\,\frac{P}{h^2}\left(\log_{10}(h^3) - 4\log_{10}\left(\sqrt{1.6\,a^2 + h^2} - 0.675\,h\right) - \log_{10} k + 6.478\right) \quad \ldots\ldots (3)$$

Note.—Written discussion on this paper will be closed in **May, 1928.**

* Presented at the Spring Meeting, Asheville, N. C., April 22, 1927.

† Pres. and Treas., Consoer, Older & Quinlan, Inc., Chicago, Ill.

‡ Fifth Annual Report, Highway Research Board, National Research Council, *Public Roads*, April, 1926.

in which, P = total wheel load; h = thickness of slab; E = modulus of elasticity of concrete (assumed hereafter to equal 3 000 000); u = Poisson's ratio of lateral expansion to longitudinal shortening (assumed hereafter to equal 0.15); a = radius of a circle over which the wheel load is assumed to be uniformly distributed; a_1 = distance from a right-angled corner to the resultant of the wheel-load pressure assumed to be applied on the diagonal at the distance, a, from either edge ($a_1 = \sqrt{2}$); and k = modulus of sub-grade reaction, assumed to be vertical only and to be proportional to the deflections of the slab. It may be stated in pounds per square inch per inch of deflection.

Choice of Constants.—The values to be assigned to both a and k seem to be worthy of consideration. It would seem that a study of the imprints of prevailing tire designs for the maximum wheel loads under consideration would permit of reasonable assumptions as to a normal value of a in a given case.

For corner loads the wheel is assumed to be placed so that the circle of equivalent loaded area is tangent to both edges. For edge loads the center of the circle is assumed to be at the edge. In any case it may be seen that the maximum stress increases quite rapidly as a approaches zero. For corner loading with $a = 0$ the stress would equal $\frac{3P}{h^2}$, and this stress would apply for the full width of any section normal to the bisector of the angle, whatever its distance from the corner. For corner, or edge, loading a might approach zero because of badly worn solid rubber tires, because of wheels largely overhanging the edge when the shoulders are soft or depressed, or because of steel tires being used. When corners are flanked by substantial flush shoulders or adjacent pavement, this condition could not well apply under rubber-tired wheels.

Curbs would be very effective in reducing stresses along outside edges. Paving widths sufficient to permit vehicles to pass readily without approaching the edge would reduce the hazard. Stone or gravel shoulders would help, although to maintain such shoulders flush with the pavement edge requires constant attention if the volume of traffic is large. In such cases, and for corner loading, it would seem proper to use for a the normal value determined by tire impression studies.

If curbs are not used and widths of, say, less than 20 ft. are contemplated, it would seem wise to use values of a of about one-fourth or less of the standard tire width for the wheel load under consideration. For edges flanked by solid shoulders or adjacent pavement a value of a somewhat larger than the normal might safely be assumed, as in such case the center of the wheel would, of necessity, be at a distance from the edge equal to one-half the tire width in order that the entire load might come upon the slab.

For interior zones, a zero value of a need hardly be considered unless provision is made for steel-tired wheels. The stress found by Equation (3) may well be replaced, as suggested by Professor Westergaard, by an equivalent one-directional stress, which he finds to be S_i $(1 - u)$. With $u = 0.15$, this would be $0.85\ S_i$. Professor Westergaard calls attention to a method of deter-

mining the value of k from deflections of existing pavements laid on similar sub-grades, and points out that values between 50 and 200 do not greatly affect stresses.

As the value of k approaches zero, however, stresses increase rapidly and there is some reason to believe that very low values of k may sometimes apply. The warping of a slab due to the rapid cooling of the top surface after sundown, following a hot sunny day, may lift corners as much as $\frac{1}{4}$ in. above the sub-grade.* Along the edges this effect is not as marked, but nevertheless may be an important factor.

With k equal to zero it may be seen that S_c (maximum stress) in Equation (1) would again equal $\frac{3P}{h^2}$. In this case, however, unless a also is very small, a stress of this magnitude would not be closely approached except at a section remote from the corner. Of necessity, the maximum tensile corner stress in slabs of ordinary dimensions must occur somewhere between the wheels of the axle. Probably with the present spacing of wheels on an axle, the distance would be not more than about 2 ft. measured along the bisector of the angle.

With values of a greater than 2 or 3 in., although k might be 0, the actual stress at 2 ft. from the corner would be substantially less than that at the section of maximum stress. For substantial values of a and small values of k the sections subject to the maximum stress would be at much greater distances from the corner than 2 ft. For corners flanked by adjacent pavement or flush shoulders capable of supporting a proportionate share of the load of an overchanging wheel, it is not probable that stresses in excess of those which would apply with normal values of a and k, would occur.

Application to Practical Design

A tabulation of stresses computed by the use of the three equations shows that if the design is such as to prevent excessive load concentrations along exposed edges, all free edges and corners will be of one thickness, the mid-portion thickness will be reduced to 0.7 or 0.75 of the edge-corner thickness, and the stresses throughout the slab for given load and sub-grade conditions will be practically equal. If, under such conditions, the value of a used in Equation (2) (for edge stress) be taken 2 in. greater than for corners and a correction for one-directional stress be made for mid-portion stress as previously discussed, the values obtained for the three positions of wheel load vary less than 10 per cent. This is shown by Table 1, in which the stresses were computed in this manner for various values of h and k and with normal a (corner) = 6 in., a (edge) = 8 in., $E = 3\,000\,000$, $u = 0.15$, and $P = 10\,000$. The mid-portion thickness is about 0.75 of the edge thickness.

The behavior of the Bates Road Test sections† seems to confirm the theory that a mid-portion ratio of 0.7 or 0.75 is justified. In these tests the same tire

* "Highway Research in Illinois" (Bates Test Road Report), by Clifford Older, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 1180.

† "Highway Research in Illinois," *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 1180.

equipment passed over both edges, the corners, and the mid-portion of all test sections and no serious damage occurred in sections of uniform thickness, except at corners. Evidently, then, the actual value of the edge stress could not have been as great or, at any rate, as serious as would be indicated by results obtained by Equation (2) (edge stress) as compared with Equation (1) using the same value of a in each case. Therefore if, in designs not permitting excessive load concentrations, all free edges are proportioned in accordance with Equation (1) and the mid-portion thickness is made 0.7 or 0.75 of the edge thickness, it would seem that no other stress computations need be made.

TABLE 1.—STRESSES IN SLAB (EQUATIONS (1), (2), AND (3)) FOR VARIOUS VALUES OF h AND k.

Thickness, h.	Modulus of sub-grade reaction, k.	STRESSES, IN POUNDS PER SQUARE INCH.		
		Corner.	Edge.	Center.
Corner and edge, 12 in.; center, 9 in..	0	208	218	210
	50	140	150	153
	100	133	138	144
	200	124	126	133
Corner and edge, 11 in.; center, 8 in..	0	248	255	258
	50	164	174	189
	100	154	160	176
	200	144	146	163
Corner and edge, 10 in.; center, 7.5 in.	0	300	301	291
	50	193	204	206
	100	182	187	192
	200	169	170	178
Corner and edge, 9 in.; center, 7 in.....	0	370	363	349
	50	233	243	237
	100	217	222	221
	200	201	200	204
Corner and edge, 8 in.; center, 6 in.....	0	469	444	434
	50	285	293	307
	100	265	266	285
	200	242	239	262

On this basis Figs. 1(a) to 1(d), inclusive, may be used to determine stresses. The diagram to be used depends on the value of a as determined by a study of tire imprints for the wheel loads to be considered. The value of k is to be determined by deflection measurements of existing slabs, as outlined by Professor Westergaard, or perhaps by noting the depression under load of small especially constructed slabs. As the stresses vary directly as the load, P, the stress for any wheel load other than 10 000 lb. may be determined from these diagrams by multiplying the reading by the ratio of actual wheel load to 10 000 lb.

Variation in Values of k.—Soil-bearing power tests conducted in Illinois on circular slabs, 3 ft. in diameter, indicate that on certain Illinois clay soils the value of k may vary from about 50 to 200 for the same soil at different seasons, depending on moisture and frost conditions. The warping of these small slabs probably did not affect the results. An examination of Fig. 1 indicates that such a variation might be disregarded for thick slabs, but that it becomes of considerable importance in designing thin pavements.

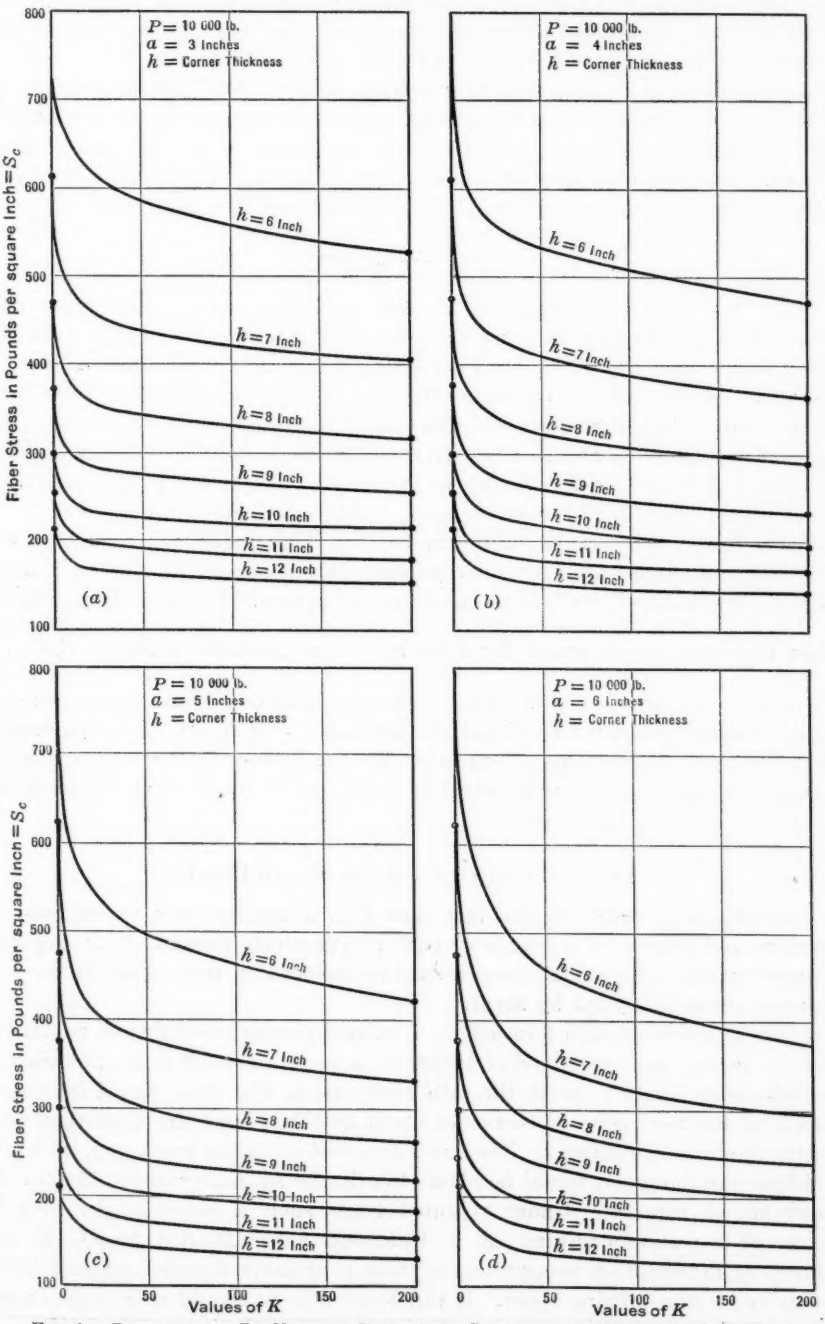

FIG. 1.—DIAGRAMS TO BE USED TO DETERMINE STRESSES IN CONCRETE PAVEMENTS.

Professor Westergaard gives equations for deflections of corners, edges, and mid-portions, and an equation for finding k therefrom. The experimental determination of the deflection of a corner or an outside edge may readily be accomplished. As a free edge may perhaps be more readily found than a free right-angled corner, only the means of computing k from edge deflections will be repeated here. With $u = 0.15$, Z_e = the deflection of the edge as determined experimentally and for a concentrated load, P, at the edge, Professor Westergaard's equations result in the following approximate expression:

$$k = 2.2 \frac{P^2}{E h^3 Z_e^2}$$

In determining the value of k from deflections of existing slabs under load, it should be kept in mind that, due to temperature warping, the deflection of a corner or edge under load at night may be several times as great as under the same load applied during the day. This is important, as the value of k determined in this manner is a function of the square of the deflection. The corner or edge must also be free. In other words, it must not be doweled to an adjacent slab, or held by friction or otherwise. The modulus of elasticity of the concrete slab should also be known within reasonable limits.

The breakage of corners on the Bates Test Road under traffic indicates very low probable values of k or a, or possibly fatigue breakage, due to repeated stresses in excess of one-half the modulus of rupture. This is indicated by the fact that stresses computed for broken corners from the formula, $S_c = \frac{3P}{h^2}$ (a or $k = 0$), averaged fairly close to the modulus of rupture of the concrete, while stresses computed by Equation (1) with $a = 4$ in. and $k = 50$, average only 64% of the modulus of rupture. It would seem that research directed toward the determination of effective values of k under various conditions would be of great importance.

Design Treatment for Joints and Cracks

Attention is called to the fact that Fig. 1 applies only to independent corners and edges. If a corner or edge be effectively doweled to an adjacent corner or edge, the actual stress would be reduced in proportion to the load carried across the joint by shear.

For a pavement with 9-in. edges, 7-in. mid-portion, modulus of rupture of 600 lb. per sq. in., wheel load of 10 000 lb., $a = 4$ in., and $k = 10$, but with an undoweled crack or joint in the thin mid-portion, Fig. 1(b) would indicate a stress at the 9-in. edge or corner of about 300 lb. per sq. in. (less than 50% of the modulus of rupture). The stress expected along the crack or joint in the mid-portion, however, would be about 465 lb. per sq. in., or about 75% of the modulus of rupture. Under repeated loads such a stress might soon be expected to result in failure. A. T. Goldbeck, Assoc. M. Am. Soc. C. E., has shown by deformation measurements* that a properly doweled center joint is effective in transmitting shear. If the doweled joint should relieve the loaded

* Fifth Annual Report, Highway Research Board, National Research Council, *Public Roads*, April, 1926.

slab only to the extent of 40%, no trouble need be expected as the inner edge stress would then be only about 280 lb. per sq. in.

Critical Conditions at Inner Points.—From this it appears evident that, in pavement design, not only must outside edges be properly provided for, but also all possible edges that may occur in the thinner mid-portion. If no transverse joints are used, contraction will open up dangerous thin sections at cracks. Expansion pressure at cracks reduces this particular hazard, but only during hot weather.

That a real danger exists in the mid-portion of a non-reinforced pavement is illustrated by the behavior of the thickened edge sections of the Bates Test Road. These sections were constructed during cold weather in the fall. Longitudinal cracking started in the thin mid-portion at six points adjacent to transverse construction joints soon after traffic began in the spring. Warm weather, however, soon placed all the transverse joints under more or less compression due to expansion, thus in effect doweling the joints. This probably accounts for the fact that no further damage occurred at these points. It is believed that trouble might have developed, had the transverse cracks not been tightly closed by expansion.

The use of transverse joints properly doweled or with thickened edges would not alone be a sufficient safeguard, inasmuch as cracks may and do appear between such joints under heavy traffic. Of the Bates Test Road 600 ft. was provided with transverse joints, at 25-ft. intervals. Under traffic, however, transverse cracks appeared in many of these sections.

Recommended Construction Practice

The following design, which is believed to take care of all conditions, is suggested:

(1) The outside edge thickness to be proportioned so that the expected stresses will be considerably less than 50% of the modulus of rupture of the concrete specified.

(2) The mid-portion thickness to be not less than 0.7 of that of the edge.

(3) The longitudinal joints to be thoroughly doweled, or provided with thickened edges if no dowels are used.

(4) Transverse joints to be installed at intervals fixed by economy. Such joints to be thoroughly doweled or provided with thickened edges.

(5) All slabs to be provided with welded steel wire mesh or closely spaced bars, the area of steel in each direction proportioned to drag one-half the width or length of the slab (while contracting), over the sub-grade, assuming a coefficient of friction of 1.5 or 2.

Reinforcement.—The use of such reinforcing steel, by holding the rough slab edges at cracks in close contact, would provide for the effective doweling of edges thus formed. It is believed that this effect would be at least equal to that attained by the best practice in center-joint construction. In such a design, assuming a properly proportioned thickness at all predetermined edges and corners, the foregoing discussion would seem to indicate that, regardless of the location of wheel loads or subsequent cracks, excessive stresses

would not occur, inasmuch as all possible interior edges would be doweled to an adjacent edge and all interior corners, to at least two adjacent corners, and indirectly to the third.

The cost of reinforcing steel proportioned in this manner increases as the transverse joint spacing increases, while the cost of joint per foot of road decreases as the spacing increases. The most economical spacing of transverse joints occurs, therefore, when the cost of the joints per foot of length of pavement, plus the cost of the reinforcing steel per foot of pavement length, is least.

Construction Details

It is obvious that, in any pavement design in which reinforcing steel is not used to relieve the slab of all tensile stresses (a condition usually forced by economy), every effort should be made to produce concrete having a high modulus of rupture and a uniform quality. It is believed that test specimens should be poured, cured with the pavement, and tested for modulus of rupture; this would give the maximum possible information as to the factors involved in producing concrete with a high modulus. The results would also be of great value in fixing the allowable stresses that may safely be used in design. Compression cylinders afford indirect evidence only of pavement strength and are of doubtful value.

Special care should be used to avoid thin spots due to high sub-grades. The design thickness should be considered as the minimum to be permitted at any point. "Stone pockets" are common in corners formed by header-boards, joints, parting strips, etc., unless special care is used in placing and spading the concrete at these points.

Slip dowels should invariably be placed parallel with the direction of expansion and with each other. They must also be provided with end pockets, if expansion joints are used.

Many other construction features of importance might be discussed, but as they are commonly included in all good modern specifications, little need be said except to emphasize the extreme importance of rigid inspection. To allow "give and take" in items affecting strength is likely to result in eventual disaster at least until it is possible to allow a more liberal factor of safety in design than is now common.

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PAPERS AND DISCUSSIONS

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THE ROLE OF THE CIVIL ENGINEER IN POWER DEVELOPMENT*

By I. W. McConnell,† M. Am. Soc. C. E.

Two papers delivered before the Power Division of the Society have presented the civil engineering features of steam power stations.‡ Both are informative in a high degree and open up a phase of engineering only too little discussed before the Society.

The author of one of the papers provoked the writer to a high degree of curiosity by intimating, with that innate modesty which has ever been the handicap of the profession, that the work of the civil engineer is the most important entering into power work. When the opportunity offered, the writer asked a steam designer and an electrical engineer how there could be any question about the matter and their joint and unanimous response was so shocking and so disrespectful that a series of studies was made by C. W. Comstock, M. Am. Soc. C. E., to see what could be revealed.

Every sermon deserves a text, so the writer took "Verily, the Civil Engineer has more opportunity to effect economies in the construction of works for the production and distribution of power than any other type of engineer connected with the design." As with every other preacher, the text was taken first and then an effort was made to prove it. This paper gives the results.

It is hoped that no one will treat any of the remarks herein in a spirit of levity. It is believed that power devices, including the emoluments and profits connected therewith, have fallen too much into the hands of mechanical engineers, electrical engineers, bankers, lawyers, promoters, horse doctors, and politicians, who regard the civil engineer as a sort of necessary evil to be employed as a hack worker to look after certain unavoidable but inconsequential details like foundations, framed structures, buildings, waterways, etc.

For many years, there has been a goodly amount of discussion in civil engineering circles as to the low rates of compensation into which civil engineer-

Note.—Written discussion on this paper will be closed in **May, 1928.**

* Presented at the meeting of the Power Division, New York, N. Y., January 20, 1927.

† Vice-Pres., Dwight P. Robinson & Co., Inc., New York, N. Y.

‡ "Civil Engineering Features of the Hell Gate Station," by Ernest M. Van Norden and George A. Hughes, Members, Am. Soc. C. E., and "The Civil Engineering Features of the Kearny Power Station," by Roman von Fabrice, Assoc. M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., March, 1917, Papers and Discussions, pp. 337 and 366, respectively.

ing has fallen and the possible remedies therefor. Having in mind that all engineering is a development from and upon civil engineering, the writer could not help but wonder, as he studied the figures which follow, whether civil engineers have improved upon their opportunities or if perchance they have allowed them to flow by and around them into the hands of experts concerned primarily with specific phases, but no more qualified at best to give a well-rounded solution to the whole problem than civil engineers.

Power is peculiarly a creation and a creature of science. The idea of its existence originated in the minds of scientists, and its evolution and development rest mainly on the efforts of those practical men of science, the engineers. The concepts of the physicist, the chemist, and the financier come back finally to the engineer for a practical assembly of constituent parts, which will translate an abstraction into an actual commodity that can be sold piecemeal for such a reasonable amount of cash as will justify its use.

Thus, it happens that the power station designer selects apparatus, such selection being mainly confined to a relatively narrow choice by the devices offered by manufacturers. He sets up an operating entity based on, and tied together by, the works of the civil engineer.

The civil engineer works in the main with such fundamentals as man-hours, cubic yards of excavation, sand, stone, gravel, lumber, cement, brick, steel, etc. No manufacturer can hand him, boxed and labeled, the devices which make up the essence of his efforts. He must build from the elements and every problem has new difficulties. Not the least of his difficulties consists in finding means by which, at a sufficiently low price to make the enterprise feasible, he can surround the space necessary to enclose the machines and devices that his brother engineers buy from manufacturers and set around in various combinations and permutations of arrangements. The writer has even the suspicion that the outdoor transformer station originated because some civil engineer previously pointed out that engineers who regard "the sky as the limit" ought to be allowed the privilege of designing their work so that it would come within and under the limit.

The gist of the whole matter is this: A successful power station is not necessarily one that turns out kilowatt-hours dependably. It is that combination of brains, capital, structures, and devices which produces a needed commodity dependably at the lowest obtainable cost, taking into account and properly compensating all the factors that enter into the creation. The last word as to undertaking an enterprise usually rests with capital. The man who satisfies the inquiries of capital has to justify his enterprise in terms of income and outgo. In the final "show-down" he rarely has to drag the boiler-room or the switching station into that final conference where the umpire says "go ahead." All such items and features are details making up the complete assembly upon which the safety of the investor's money depends. The writer hopes that civil engineers may take a larger part in the economics and the underlying business reasons involved in power production and sale and not be confined to the smaller economics of the strictly civil engineering features.

Notwithstanding the multiplicity of special engineers, some of whom are described by fantastic adjectives, there remains a great range of work which,

by common consent, lies within the province of the old-fashioned civil engineer. Such are foundations, extensive excavations, structures of steel, masonry, and concrete, works for the control, diversion, and regulation of water, railroads, highways, and, in general, those constructions which are mostly stationary as distinguished from machines in the ordinary sense. A considerable proportion, much larger than is generally thought, of the works necessary for power development and distribution fall within the scope of the civil engineer in the narrow meaning of present-day practice.

Large scale power development in stationary plants may be broadly classified as (1) hydro-electric and (2) steam. With respect to the first it is nearly impossible to generalize. Such basic considerations as head, volume of water, storage or pondage possibilities, topographical situation, foundation conditions, etc., vary over a wide range, and no two plants are, or can be, even approximately alike. Although in every instance the total cost per unit of capacity must fall within an established limit, the distribution among the various elements of the installation cannot be inferred from one plant to another.

Tables 1 to 5, inclusive, show the cost distribution among the principal features of a number of hydro-electric plants in various parts of the United States.

TABLE 1.—HYDRO-ELECTRIC PLANT NO. 1.
(About 20 000 kw.)

Item.	Percentage of cost.
Intake, flow line, surge tank, penstocks, and tail-race	41.82
Generating station and switchyard	7.67
Hydraulic, generating, and electrical equipment	18.07
Miscellaneous	4.74
Construction plant	10.30
Engineering	4.80
Real estate, general expenses, and interest during construction	12.60
Total	100.00

With the exception of 30.67% for machinery, real estate, interest and general expenses, this installation was entirely the work of the civil engineer.

TABLE 2.—HYDRO-ELECTRIC PLANT NO. 2.
(About 20 000 kw.)

Item.	Percentage of cost.
Dam, intake, flow line, surge tank, penstocks, and tail-race	38.70
Power-house building and switchyard	7.90
Hydraulic and electrical equipment	24.80
Miscellaneous	2.50
Construction plant	6.90
Engineering	7.10
General expenses and interest during construction	12.60
Total	100.00

TABLE 3.—HYDRO-ELECTRIC PLANT NO. 3.
(About 25 000 kw.)

Item.	Percentage of cost.
Dam, forebay, canal, power-house substructure, and tail-race	48.10
Power-house superstructure	4.40
Power-plant equipment	20.60
Miscellaneous	3.50
Construction plant	4.40
Engineering	5.60
General expenses and interest during construction	13.40
Total	100.00

TABLE 4.—HYDRO-ELECTRIC PLANT NO. 4.
(About 7 000 kw.)

Item.	Percentage of cost.
Coffer-dams, excavation, permanent dam, and spillway	29.60
Power-house	9.70
Power-plant equipment	41.40
Miscellaneous	7.30
Construction plant	1.70
Superintendence	3.10
General expenses	2.70
Contractor's profit	4.50
Total	100.00

TABLE 5.—HYDRO-ELECTRIC PLANT NO. 5.
(About 10 000 kw.)

Item.	Percentage of cost.
Dam, head-works, tunnel, forebay, spillway, and penstock	57.78
Power-house	4.51
Power-plant equipment	7.51
Miscellaneous	4.50
Construction plant	11.28
Engineering	3 38
Superintendence	1.08
General expenses	9.96
Total	100.00

It is commonly assumed by both engineers and laymen that the problem of the steam power station belongs solely to the province of the mechanical engineer or to that of the steam power specialist.

The civil engineer would be the first to say that steam power is out of his line. However, the costs of some large steam stations have been analyzed in order to determine to what extent the total cost might be influenced by skillful civil engineering design. The results are summarized in Tables 6 to 8, inclusive.

TABLE 6.—STEAM PLANT No. 1.
(Total cost, $14 000 000.)

Item.	Percentage of total cost.
Dredging, dikes, and seawall	1.01
Station yard	0.75
Tracks, trestles, scales, etc	1.48
Building	17.58
Machinery foundations	0.52
Stacks and stack lining	1.38
Condenser system	2.82
Outside switching station	0.80
Coal storage, etc	0.40
All civil engineering features	26.74

TABLE 7.—STEAM PLANT No. 2.
(Total cost, $4 250 000.)

Item.	Percentage of total cost.
Clearing, grading, and preparatory	0.67
Tracks, trestles, etc	0.54
Building	19.34
Machinery foundations	0.42
Stacks and stack lining	0.94
Condenser system	2.87
Outside switching station	0.51
Coal and ash-handling	0.54
Water supply lines	1.94
All civil engineering features	27.77

TABLE 8.—STEAM PLANT No. 3.
(Total cost, $4 250 000.)

Item.	Percentage of total cost.
Station yard	1.53
Tracks	0.08
Building	16.56
Machinery foundations	0.54
Stack	0.74
Condenser system	4.70
Oil storage and handling	0.33
All civil engineering features	24.48

In a small station costing about $600 000, the civil engineering features aggregated 17% of the total. In this instance no boiler plant was constructed, and the turbine room was merely the extension of an existing building.

An illustration cited in the technical press of the importance of skillful structural design is to be found in the Richmond Station of the Philadelphia Electric Company, put into operation in November, 1925. The structural

framework of this station contains 11 000 tons of steel, of which 8 000 tons is silicon steel permitting a unit stress of 22 000 lb. per sq. in., instead of the usual 16 000 lb. Thus, the frame weighs 3 000 tons less than it would have, if ordinary soft structural steel had been used throughout, a saving of more than 20% in weight.

Attention is now called to an analysis of the cost figures of a plant which will be called "X". It represents the beginning of a large station to contain more than one generating unit. The building was constructed for two units of 60 000 kw. each, and the mechanical and electrical equipment were laid out for one unit only, but with provision for expansion as the demand increased. The figures presented do not represent the true relation in a complete station as between the classes of engineering; but they happen to be the most complete available to show the relation between work designed and manufactured articles bought, and in that respect they do show a true relation. These figures are as follows:

Total outlay $8 500 000

Distribution:	Percentage of total cost
Overhead	9.5
Civil engineers.....	40.0
Manufactured goods purchased....	2.0
Mechanical engineers	40.0
Manufactured goods purchased....	30.0
Electrical engineers	8.5
Manufactured goods purchased....	5.0

Thus, the civil engineer buys 5% of his work, the mechanical engineer buys 66% of his, and the electrical engineer buys nearly 60% of his.

Careless, incompetent, or extravagant design may easily increase the cost of foundations, buildings, and other purely structural elements by 20% of the proper minimum. This means 5% added to the cost of the complete plant and an equal percentage added to fixed charges which must be incorporated in service rates. The writer ventures the suggestion that it will be less costly and less troublesome to employ competent civil engineers to save that 5% in fixed charges than to pay corporation lawyers to justify a 5% increase in rates. So much for the influence of a structural design on first cost.

Can the Civil Engineer influence operating costs? The elements of the cost of steam power in a large plant operating at about 60% load factor are approximately as follows:

	Percentage
Fixed charges	48
Fuel	36
Operation and maintenance.....	16

The effect of structural design on fixed charges has already been discussed. Fuel consumption can be influenced by the civil engineer only in the matter of power consumed by auxiliaries, notably by circulating pumps. Most of the power consumed in the circulation of condensing water goes to overcome the

various hydraulic resistances of the inlet and discharge tunnels. Here, the skill of the hydraulic engineer may be applied to good purpose. The percentage effect on the coal pile is small, but the aggregate in dollars and cents in a large plant is a respectable sum. Fig. 1,* showing the tunnel layout at the Richmond Station of The Philadelphia Electric Company, will give some idea of the importance of skillful design of this feature.

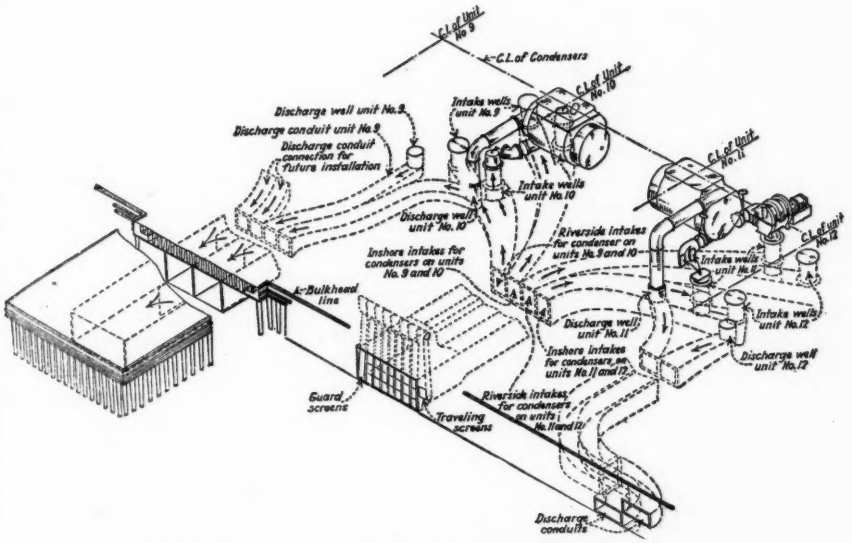
C.L. of Unit No. 9
C.L. of Condensers
C.L. of Unit No. 10
C.L. of Unit No. 11
C.L. of Unit No. 12
Discharge wall unit No. 9
Discharge conduit unit No. 9
Discharge conduit connection for future installation
Intake wells unit No. 9
Intake wells unit No. 10
Intake wells unit No. 11
Intake wells unit No. 12
Discharge wall unit No. 10
Riverside intakes for condenser on units No. 9 and 10
Inshore intakes for condensers on units No. 9 and 10
C.L. Bulkhead line
Discharge wall unit No. 11
Inshore intakes for condensers on units No. 11 and 12
Discharge wall unit No. 12
Guard screens
Traveling screens
Riverside intakes for condensers on units No. 11 and 12
Discharge conduits

FIG. 1.—ARRANGEMENT OF INTAKE AND DISCHARGE TUNNELS.

Of the 16% chargeable to operation and maintenance, only that portion relating to the maintenance of structural features can be affected by civil engineering design. This must represent only a very small percentage of the total, but the saving in dollars may be quite considerable.

Relative to power transmission, transmission line design and construction is almost entirely a matter of civil engineering from the location to the provision for mechanical stresses. These are the determining features. On the choice of span lengths, design of towers, etc., the cost is almost wholly dependent. The electrical engineer fixes the requisite electrical properties of conductors and insulators; beyond this, he is without influence on the cost of the line.

Table 9 shows the cost distribution on high tension lines about 125 miles long. It is evident that more than three-quarters of the cost is markedly influenced by location, choice of spans, design of structures, and general administration. No more could be said of a bridge, railroad, highway, or any other admittedly civil engineering work. Corresponding details for other transmission lines are given in Tables 10 to 15, inclusive.

* Reproduced from *Power*, May 18, 1926.

TABLE 9.—Transmission Line No. 1.

(High-Tension Line About 125 Miles Long.)

Item.	Percentage of cost.
Towers and special structures	25.10
Conductors	40.60
Insulators	6.30
Ground wire	2.50
Surveying and right of way	5.00
Construction plant and miscellaneous	2.80
Engineering	7.30
General expenses and interest during construction	10.40
Total	100.00

TABLE 10.—Transmission Line No. 2.

(About 15 Miles Long.)

Item.	Percentage of cost.
Towers and special structures	29.80
Conductors	38.50
Insulators	4.20
Ground wire	3.00
Surveying and right of way	4.10
Construction plant and miscellaneous	3.50
Engineering	7.30
General expenses and interest during construction	9.60
Total	100.00

TABLE 11.—Transmission Line No. 3.

(About 25 Miles Long.)

Item.	Percentage of cost.
Towers and special structures	22.40
Conductors	30.60
Insulators	6.30
Ground wire	1.60
Surveying and right of way	11.20
Construction equipment and miscellaneous	5.70
Engineering and inspection	7.10
Superintendence	2.40
Telephone line	6.60
General expenses and interest during construction	6.10
Total	100.00

TABLE 12.—Transmission Line No. 4.

(About 5 Miles Long.)

Item.	Percentage of cost.
Towers and special structures	22.10
Conductors	24.10
Insulators	6.90
Ground wire	1.30
Surveying and right of way	27.00
Construction plant	0.60
Engineering	8.30
General expenses and interest during construction	9.70
Total	100.00

TABLE 13.—Transmission Line No. 5.

(Wood Pole Line About 15 Miles Long.)

Item.	Percentage of cost.
Poles, cross-arms, hardware, and special structures	28.00
Conductors	28.60
Insulators	3.10
Ground wire	0.90
Surveying and right of way	12.50
Construction plant	3.00
Engineering	8.90
Telephone circuit	2.50
General expenses and interest during construction	12.50
Total	100.00

TABLE 14.—Transmission Line No. 6.

(Wood Pole Line About 60 Miles Long.)

Item.	Percentage of cost.
Poles, cross-arms, hardware, and special structures	18.90
Conductors	31.00
Insulators	8.40
Ground wire	3.00
Surveying and right of way	8.70
Construction plant and miscellaneous	8.00
Engineering	7.20
Telephone line	4.40
General expenses and interest during construction	10.40
Total	100.00

TABLE 15.—TRANSMISSION LINE No. 7.

(Wood Pole Line About 30 Miles Long.)

Item.	Percentage of cost.
Poles, cross-arms, hardware, and special structures	17.50
Conductors	32.80
Insulators	7.60
Ground wire	3.10
Surveying and right of way	9.10
Construction plant and miscellaneous	7.70
Engineering	7.30
Telephone line	4.60
General expenses and interest during construction	10.30
Total	100.00

The foregoing is intended as an exposition, in a practical way, of the extensive participation of the civil engineer in power development. The opportunity is only indicated by these tables and examples. The greatest element of cost of power to the small consumer is the cost of distribution and, in that phase of the business, the civil engineer will find numerous opportunities. In the cultivation of commercial acumen as an adornment to technical excellence the civil engineer may well contemplate the power field.

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FLOOD CONTROL WITH SPECIAL REFERENCE TO THE MISSISSIPPI RIVER

A SYMPOSIUM*

THE EFFECT OF AGRICULTURAL DRAINAGE UPON FLOOD RUN-OFF

By Sherman M. Woodward† and Floyd A. Nagler,‡ Members, Am. Soc. C. E.

Synopsis

A survey of completed drainage enterprises in the Upper Mississippi Valley indicated that a comparison of stream-flow conditions prior and subsequent to extensive drainage could best be made upon the Des Moines and Iowa Rivers, a large portion of the water-sheds of which have been covered with artificial drains subsequent to the establishment of stream-gauging stations.

A critical examination of the records of these two streams shows that during flood periods there has been no significant change in their behavior which may be attributed to drainage. The total run-off from storms of like precipitation, the maximum rates of discharge, and the rain-water storage conditions within the basins seem to have been unaltered by the extensive drainage operations. It is believed that if any of these factors had been changed by a measurable amount, such fact could easily have been detected by the analysis made in this paper.

The drainage operations on the water-sheds of these two rivers involved the construction of tile drains, open ditches, and some straightening of stream channels, typical operations in drainage projects in the Upper Mississippi

Note.—Discussion on this Symposium will be closed in April, 1928.

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Valley. Since there has been no appreciable change in the run-off from these water-sheds which have experienced extensive drainage, it appears, therefore, that agricultural drainage has had a negligible effect upon the magnitude of either the total flow or the maximum discharge of the floods of the Mississippi River, less than 5% of the water-shed of which has witnessed agricultural drainage operations.

Although many references have been made in recent years to the effect of agricultural drainage upon flood run-off in general, and numerous statements have been published regarding the relation of such drainage to the magnitude and frequency of floods in the Mississippi and other rivers, almost no actual numerical evidence has hitherto been brought forward tending definitely to prove or disprove any of the relationships alleged.

This paper is a painstaking effort to determine whether existing records indicate the presence of any relationship or no relationship between agricultural drainage and flood run-off. All the data shown in this paper are based upon official Government records; the data concerning drainage are abstracted from the Fourteenth Census of the United States for 1920, Volume VII, Irrigation and Drainage; the data concerning run-off are taken from the *Water Supply Papers* of the U. S. Geological Survey; and the data on rainfall from the publications of the U. S. Weather Bureau.

In the Census report a distinction is made between farm drainage on the one hand, that is, the small-scale drainage, such as tile drainage, for example, that can be carried out by an individual farmer independent of any action by his neighbors, and drainage enterprises on the other hand, those larger cooperative undertakings requiring the joint effort of more than one land owner to consummate the community enterprise. As all familiar with agricultural drainage on an extensive scale will know, this latter class of drainage work is much the more extensive of the two and is the class that may be expected to have the chief effect upon stream run-off. All the subsequent references to drainage contained herein pertain solely to this second class. Table 30 presents a summary of statistics on drainage operations within the United States and those portions of the Mississippi River water-shed that were intensively studied in this investigation.

Fig. 29 is a diagram showing the rate of increase in the area embraced in drainage enterprises in the whole United States during the last fifty years. It will be noted that the rate of progress was relatively slow until 1900, and that its greatest rate of increase was between 1905 and 1915.

A map of the United States published in the 1920 Census report,* shows all the area included on January 1, 1920, in operating drainage enterprises, that is, enterprises the construction work of which had either been completed or begun. Of the total area included in these enterprises, about one-half is embraced in the Mississippi Valley. According to the Census figures between 4 and 5% of the total Mississippi River drainage area is included in operating drainage enterprises. Most of this is in the northern part of the country,

* U. S. Census Rept., 1920, Pt. VII, p. 347.

that is, in the areas tributary to the Missouri, the Upper Mississippi, and the Ohio Rivers. Two large, compact, drained areas stand out conspicuously, the more eastern one in Ohio, Indiana, and Illinois, the other in North Central Iowa.

TABLE 30.—DRAINAGE STATISTICS FROM 1920 CENSUS REPORT.*

(*A*) DRAINAGE STATISTICS FOR UNITED STATES AND IOWA.

	In United States, in acres.	In State of Iowa, in acres.	Page reference in U. S. Census Report, 1920.
In operating enterprises	65 495 038	5 224 478	373
In non-operating enterprises	3 924 821	158 534	373
Completed in 1920	56 763 751	4 685 080	373
By open ditches only	43 658 485	1 294 243	376–377
By open ditches and levees	5 656 000	26 228	376–377
By only tile drains	4 951 152	1 415 176	376–377
By open ditches and tile drains	10 731 279	2 444 906	376–377
By ditches, tile, and levees	475 013	37 925	376–377

(*B*) DRAINAGE STATISTICS FOR MISSISSIPPI RIVER BASIN.

	Area in operating enterprises, in acres.	In non-operating enterprises, in acres.	Total, in acres.	Page reference in U. S. Census Report, 1920.
Upper Mississippi River	10 850 408	521 759	11 372 167	367
Missouri River	2 510 150	352 894	2 863 044	367
Ohio River	10 998 378	158 932	11 157 310	367
Lower Mississippi River	8 072 564	1 194 600	9 267 164	367
Total	32 431 500	2 228 185	34 659 685	.....

(*C*) DRAINAGE STATISTICS FOR DES MOINES and IOWA RIVER BASINS.

	Des Moines River Basin.		Iowa River Basin.		Page reference in U. S. Census Report, 1920.
	Acres.	Square miles.	Acres.	Square miles.	
In operating enterprises	2 808 170	4 388	936 841	1 461	481 541
In non-operating enterprises	96 987	152	0	0	481
Total	2 905 157	4 540	936 841	1 461	.....

*Vol. VII, Irrigation and Drainage.

A study of the records shows that in the more easterly area much of the drainage work antedated extensive reliable stream measurements, thus precluding a satisfactory comparison of stream flow before and after drainage. A more satisfactory condition exists with reference to the Iowa area. In this region drainage work was not extensive until after the passage of the 1904 Drainage Law, but nearly all the work was completed by 1917. The greater part of this compact area of drained Iowa land lies in the upper portions of

the drainage areas of the Des Moines and the Iowa Rivers. Most fortunately for the purpose of this paper, stream measurements were begun upon these rivers by the U. S. Geological Survey in 1903. The progress of drainage in Iowa is shown in Fig. 30. For successive dates, the full line curve shows the area, in millions of acres, contained in authorized drainage enterprises; the broken lines show similarly the thousands of miles of tile drains and open ditches.

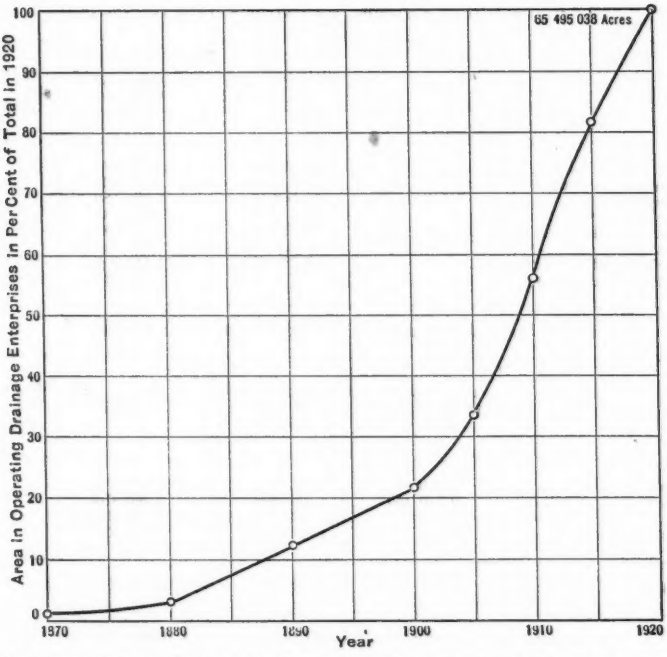

FIG. 29.—PROGRESS OF DRAINAGE IN THE UNITED STATES. FROM CENSUS OF 1920.

After a careful scrutiny of the situation it was decided to be best to study the stream-flow records for a four-year period, from January 1, 1903, to December 31, 1906, to ascertain the behavior of these rivers prior to drainage, and to use the records similarly for the six-year period, January 1, 1918, to December 31, 1923, to learn their behavior subsequent to extensive drainage operations. Apparently, nowhere else in the United States are conditions so favorable for making this kind of comparison.

On the Iowa River a record of discharge has been maintained since 1903 at Iowa City, Iowa, above which point the drainage area is 3 140 sq. miles. Of this area, more than one-third is in operating drainage enterprises.

Upon the Des Moines River three gauging stations were started before the period of greatest drainage activity, and have been maintained to date with some fragmentary intermissions. The uppermost station was established near Fort Dodge, Iowa, with a drainage area of 4 170 sq. miles, nearly all of which has been embraced in drainage enterprises. The second gauging station is at Des Moines, Iowa, with a tributary area of 6 180 sq. miles, about two-thirds

of which has been drained. The lowest station is at Keosauqua, Iowa, embracing 13 900 sq. miles, more than one-third of which has been artificially drained.

Fig. 31 shows on an outline map of Iowa the position of the drainage areas of the Iowa and Des Moines Rivers, and the locations of the gauging stations. The drainage areas are long and narrow, that of the Des Moines extending into Minnesota. The shaded area is that portion embraced in drainage enterprises. The map also shows the isohyetal lines or rainfall contours for what is subsequently called Storm No. 1, lasting from May 8 to June 26, 1903. This storm was unusually heavy, the greatest in the four-year pre-drainage period, and will be further referred to later. The dots on the map show rainfall stations used in subsequent studies.

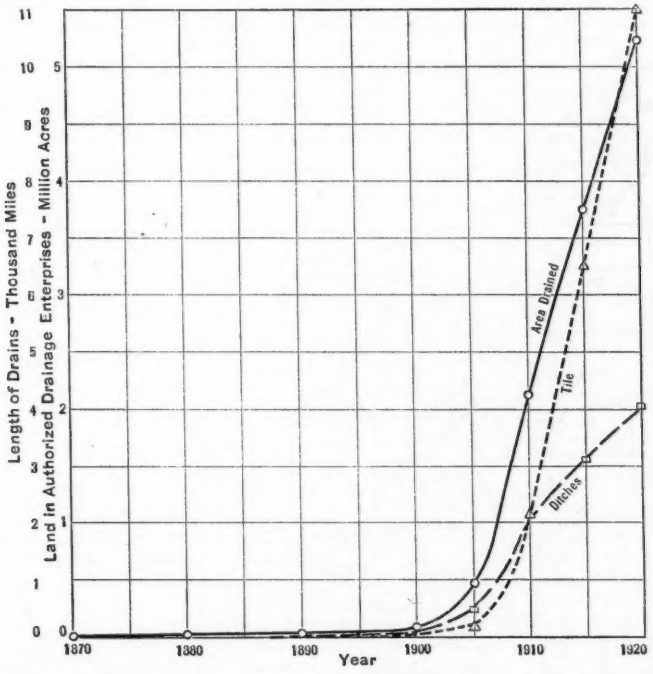

FIG. 30.—PROGRESS OF DRAINAGE IN IOWA. FROM U. S. CENSUS OF 1920.

The northern part of these drainage areas is a flat prairie, with scanty, natural, shallow, drainage channels. Before artificial drainage a large portion of the land was in marshes or too wet in the spring for profitable cultivation. The lower part of the drainage areas is rolling, with the highest portions of the land nearly 100 ft. above the rivers. Originally, most of this lower part was covered with hard-wood timber, but the greater portion of this has long been removed. The drainage improvements have included the construction of open ditches and tile drains as well as the deepening and straightening of natural channels.

Iowa streams yield in run-off less than one-quarter of the water which they receive in the form of precipitation. About three-quarters of this annual run-

off is discharged during periods of high water, and it is the high-water periods only that make any significant contributions to Mississippi River floods. If the carrying out of agricultural drainage affects the flow of streams in any measurable way, such effects must be most conspicuous near the drained areas. More remote points down stream would naturally be less affected, in spite of any cumulative result from many tributaries. If no effect can be perceived in the streams traversing the drained areas, it may reasonably be concluded that no similar effects are produced at remote points on the larger rivers.

FIG. 31.—DRAINAGE AREAS OF DES MOINES AND IOWA RIVERS. STORM NO. 1, MAY 8 TO JUNE 26, 1903.

As a first preliminary study, the monthly run-off was compared with the monthly precipitation during the spring months of March, April, and May, and all other single months in which the stream flow was exceptionally large during the adopted pre-drainage and post-drainage periods. In this study, the precipitation was computed by taking the arithmetical average of that recorded at selected rainfall stations distributed over each basin so that each station represented an area of about 800 sq. miles. On the Iowa River five

stations were used; on the Des Moines, five were used above Fort Dodge, seven above Des Moines, and fourteen above Keosauqua. These values of monthly precipitation and run-off are shown in Table 31 and in Fig. 32, in order that the stream behavior prior to drainage may be compared with that subsequent to drainage.

TABLE 31.—MONTHLY PRECIPITATION AND RUN-OFF ON DES MOINES AND IOWA RIVERS.

Year.	Month.	Des Moines River.						Iowa River.	
		Above Fort Dodge.		Above Des Moines.		Above Keosauqua.		Above Iowa City.	
		Precipitation, in inches.	Run-off, in inches.	Precipitation, in inches.	Run-off, in inches.	Precipitation, in inches.	Run-off, in inches.	Precipitation, in inches.	Run-off, in inches.
BEFORE DRAINAGE.									
1903	March			0.95	1.02				
	April			2.69	0.67				
	May			9.96	1.88	9.65		7.39	
	June			3.29	1.81	3.63	2.69	2.45	2.59
1904	March								
	April					4.37	1.25	3.54	0.82
	May					3.22	0.61	3.69	0.58
1905	March					2.12	1.03	2.12	1.35
	April					3.42	0.66	2.91	0.70
	May	7.37	0.99			5.76	1.04	7.12	1.73
	June	4.58	0.71	5.08	0.75	5.66	1.00	4.59	1.08
1906	February					1.28	0.59		
	March			2.33	0.89	2.12	0.86	2.60	1.02
	April	1.65	1.08	2.08	1.09	2.40	1.14	2 29	1.69
	May	4.42	0.64	3.92	0.64	3.18	0.52	3.08	0.64
AFTER DRAINAGE.									
1918	March								
	April	1.54	0.15	1.58	0.15	1.76	0.10	2.70	0.23
	May	7.43	0.28	6.92	0.27	6.41	0.31	8.32	1.30
	June	4.53	0.82	5.34	1.75	5.08	1.35	7.53	3.90
1919	March			1.83	0.86	2.20	1.04	2.27	1.46
	April	4.78	1.68	5.15	1.81	5.14	1.45	5.14	1.70
	May	2.17	1.12	2.21	1.21	2.62	1 35	3.07	1.83
	June	6.47	1.68	6.67	1.60	7.05	1.67	6.43	1.74
	July	3.62	0.80	3.28	0.68	3.41	0.59	1.68	0.47
1920	March	2.21	1.13	2.37	1.03	2.52	1.28	3.30	1.52
	April	3.81	0.77	3.84	0.84	4.31	1.24	4.31	1.50
	May	2.29	1.05	2.37	1.08	2.67	1.27	3.65	1.54
	July	5.92	1.83	6.01	1.49	5.35	1.08	4.03	0.55
1921	February	0.97	0.42	0.99	0.49	0.96	0.39	0.89	0.42
	March	1.30	0.40	1.24	0.38	1.36	0.31	1.49	0.42
	April	2.01	0.39	2.20	0.44	2.84	0.57	4.11	0.63
	May	7.17	0.77	6.03	0.71	4.89	0.61	4.31	0.73
	September	5.74	0 37	6.03	0.66	6.50	0.87	7.79	0.53
1922	March	1.17	0.60	1.40	0.52	2.01	0.63	1.91	0.98
	April	3.74	0.90	3.53	0.95	3.20	1.09	3.09	1.20
	May	0.88	0.31	2 90	0.42	3.57	0.57	4.11	0.58
1923	March	1.56	0.30	2.00	0.39	2.54	0.62	3.25	0.54
	April	2.29	0.24	2.14	0.38	1.88	0.48	1.60	1.26
	May	2.59	0.12	3.06	0.21	3.01	0.31	2.48	0.28

The quantity of water appearing as run-off from a given amount of precipitation is no doubt larger during the early spring months than later, de-

creasing with the increase in temperature and the growth of vegetation. This fact was taken into consideration by plotting the monthly values for each gauging station in three separate groups. One group includes the records for March and a few for February; the second group includes those for the month of April; and the third group includes those for the months of May, and a few for the months of June, July, and September. In this, as in subsequent diagrams, the values of run-off prior to drainage, have been indicated by solid symbols, in contrast to the open symbols used for the post-drainage period.

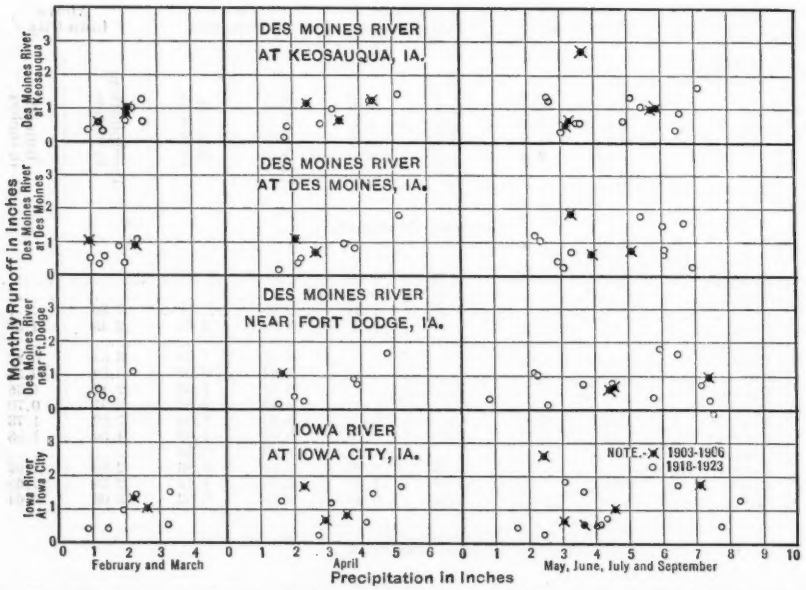

FIG. 32.—RELATION OF PRECIPITATION TO RUN-OFF IN MONTHS OF HIGH WATER ON THE DES MOINES AND IOWA RIVERS, COMPARING 1903–1906 WITH 1918–1923.

The plotting of data of this kind naturally results in a wide scattering of points, because the run-off for a given month may have been the result of precipitation which fell during the previous month, to say nothing of the various hydrologic factors which may have caused variations in the stream yield. The points showing the run-off previous to drainage seem to group themselves well within and about the system of points obtained for the post-drainage period, and no other conclusion is possible than that the average monthly yield of the streams subsequent to drainage does not seem to be different from that previous to drainage. If any difference can be detected from this plotting, it would appear to indicate that the streams yielded a greater quantity during the flood months of 1903 to 1906 than during those of 1918 to 1923. Certainly, other factors than drainage have been more significant in producing differences in stream yield during both the pre-drainage and the post-drainage period.

In order to make a more delicate and conclusive comparison of the behavior of the streams before and after drainage than that presented in Fig. 32, every flood of significance which occurred on these streams during these periods, was next studied in considerable detail. Each flood that produced a maximum run-off greater than 2 sec-ft. per sq. mile was included in this investigation. The total run-off was computed from the beginning of the flood until the dis-

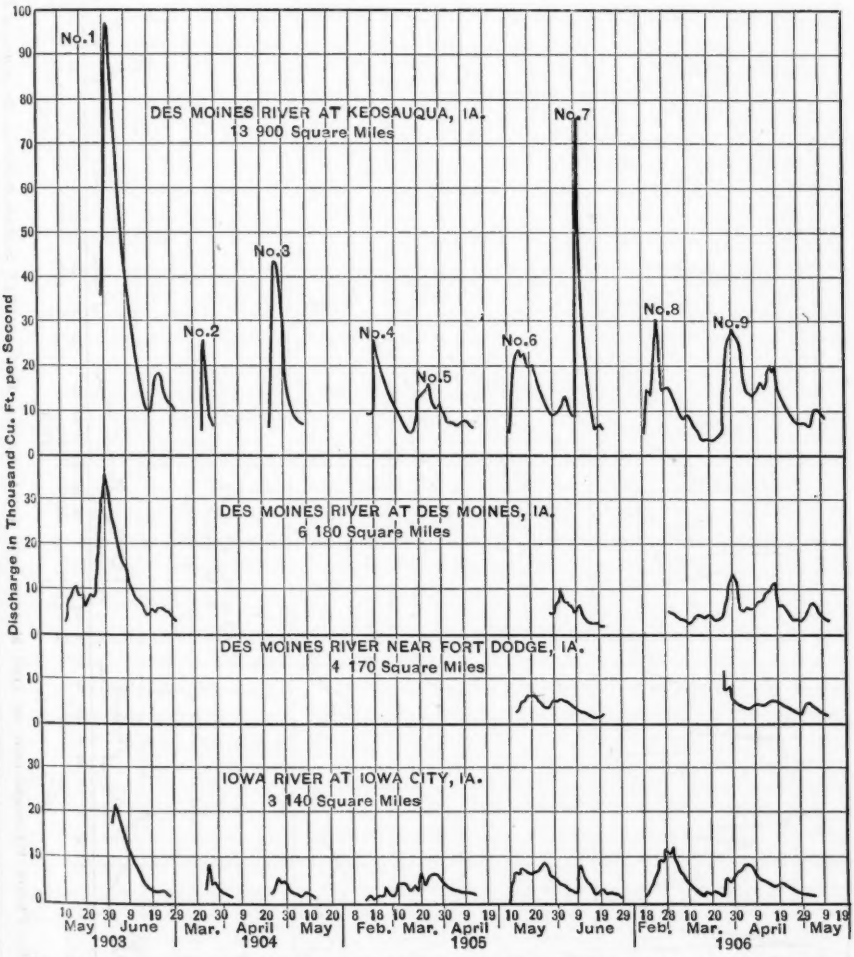

FIG. 33.—FLOOD HYDROGRAPHS ON DES MOINES AND IOWA RIVERS, 1903 TO 1906, PREVIOUS TO EXTENSIVE DRAINAGE.

charge had diminished to a flow of ½ sec-ft. per sq. mile, or lower, and the average precipitation producing this high water was evaluated from all rainfall stations within and about the water-sheds in question. Nine such floods occurred during the pre-drainage period, 1903 to 1906, and eleven during the post-drainage years, 1918 to 1923. Individual hydrographs of these floods are shown in Figs. 33 and 34.

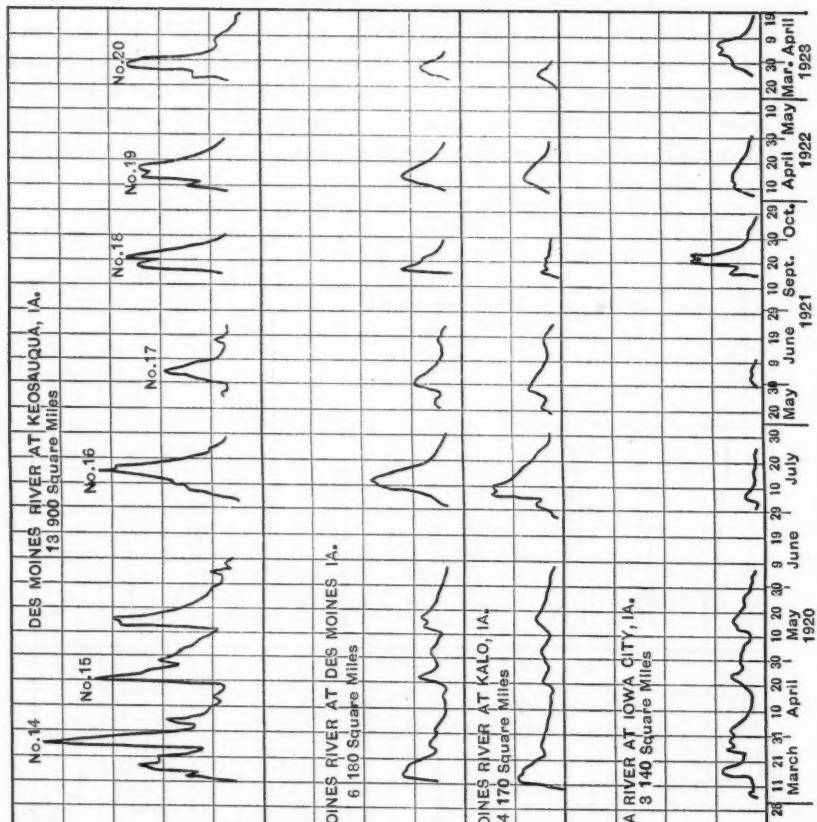

FIG. 34.—FLOOD HYDROGRAPHS ON DES MOINES AND IOWA RIVERS, 1918 TO 1923, SUBSEQUENT TO EXTENSIVE DRAINAGE.

The average precipitation was computed by applying the method suggested by Thiessen* to obtain the true weighted average precipitation on the water-shed. All available precipitation stations were used, the number considered for each water-shed varying somewhat from year to year. The maximum and minimum numbers were as given in Table 32.

TABLE 32.

Water-shed.	Number of Precipitation Stations.	
	Maximum.	Minimum.
Des Moines River:		
Above Fort Dodge	13	9
Above Des Moines	21	17
Above Keosauqua	45	30
Iowa River:		
Above Iowa City	16	12

Isohyetal lines were also drawn, for each stormy period, upon maps of the drainage area in order to visualize the distribution of the storm. Other values pertinent to the study of these storms and their yield have been given in Tables 33 and 34.

In order to study the amount of run-off produced by a given amount of "effective precipitation" these quantities for each storm period were plotted on Fig. 35, employing the same general scheme of symbols used previously in Fig. 32. In order to facilitate this study these storms were first classified into three groups according to the average temperature which existed during the storm period. One group, with average temperatures below 45°, includes all early spring storms coming before the growing season had started; a second group, with average temperatures between 45° and 60°, includes those storms which occurred during the early growing season; the third group, with average temperatures exceeding 60°, includes the early and late summer storms which occurred in June, July, and September, the storms which generally include the most intense rainfall and produce the highest floods on these streams. Some distinction was made in plotting the run-off from these storms, depending upon the location of the greatest amount of rainfall on the basin. Triangles are used to indicate the storms in which the heaviest rainfall occurred on the upper part of the drainage area; circles to indicate storms near the center, or spread uniformly over the whole drainage area; and squares to indicate storms heaviest on the lower portion of the basin. The lines showing the average relation between rainfall and run-off were drawn from the point of zero rainfall and precipitation through the area covered by the open circles, representing those storms which were of a general nature or concentrated about the center of the basin. It was noted in plotting the data on storms subsequent to drainage that those in the upper basin gave a lower run-off than general storms (indicated by the circles), and that storms in the lower basin gave a higher run-off.

* *Journal*, New England Water Works Assoc., March, 1924, p. 26.

TABLE 33.—SUMMARY OF FLOODS ON DES MOINES RIVER.

Flood No.	Year.	Date of Storm. Time. Beginning.	End.	Duration of storm, in days.	Number of stormy periods.	Number of days of more than 0.25 in. of rainfall.	Total effective rainfall, all days, with excess of 0.25 in.	Total rainfall, in 30 days before storm.	Date of maximum rainfall.	Maximum 24-hour rainfall.	Location of storm center.	Average temperature at Des Moines during storm, in degrees Fahrenheit.	Time of Flood. Beginning.	End.	Duration of flood, in days.	Total run-off during flood, in inches.	Date of maximum flow.	Maximum 24-hour flow, in second-feet.	Ratio, run-off to rainfall.	Average rate of rainfall, in inches per day.	Ratio, maximum flow to average flow.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
FLOODS ON DES MOINES RIVER ABOVE FORT DODGE, IOWA—PERIOD BEFORE DRAINAGE.																					
6	1905	May 9	June 7	30	4	6 to 7	6.81	1.66	May 10	2.20	*GU*	62	May 12	June 9	29	1.24	May 17–18	6 460	0.182	0.221	1.34
7	1905	June 8	June 21	14	2	2 to 4	1.63	6.71	June 18	1.77	*L*	71	June 10	June 21	12	0.20	June 8	3 450	0.123	0.136	1.81
9	1906	Mar. 22	May 4	44	5	1 to 8	2.68	0.96	May 1	2.00	*U*	50	Mar. 26	May 10	47	1.74	Mar. 26	12 000	0.650	0.061	2.85
FLOODS ON DES MOINES RIVER ABOVE FORT DODGE, IOWA—PERIOD AFTER DRAINAGE.																					
10	1918	May 26	June 7	13	2	4 to 6	4.58	4.69	June 4	3.35	*MU*	70	May 28	June 13	17	0.78	June 4	11 400	0.172	0.348	2.20
11	1919	Mar. 9	Apr. 5	28	3	1 to 3	0.96	1.66	Mar. 15	1.33	*L*	45	Mar. 13	Apr. 6	25	1.16*	Mar. 17–20	7 330	1.21	0.034	1.41
12	1919	Apr. 6	May 31	56	6	6 to 12	5.27	1.60	Apr. 7	2.10	*GL*	55	Apr. 7	May 31	55	2.58	Apr. 21	7 970	0.490	0.094	1.50
13	1919	June 1	July 18	48	8	9 to 12	8.75	1.89	July 4	3.10	*MU*	75	June 1	July 21	51	2.35	June 24–25	10 200	0.268	0.182	1.96
14	1920	Mar. 10	Apr. 12	34	8	2 to 6	2.44	0.91	Mar. 11	1.45	*L*	43	Mar. 10	Apr. 17	39	1.50	Mar. 13–14	9 000	0.615	0.072	2.09
15	1920	Apr. 14	June 7	55	10	3 to 11	5.65	2.93	Apr. 19	1.63	*G*	58	Apr. 18	June 7	51	1.57	May 17	5 350	0.278	0.103	1.57
16	1920	June 27	July 27	31	5	7 to 10	6.30	2.17	July 6	2.30	*UL*	74	June 29	July 30	32	1.86	July 7–8	13 700	0.295	0.208	2.11
17	1921	May 16	June 22	38	5	5 to 11	6.28	2.49	May 26	2.79	*U*	75	May 19	June 22	35	1.18	May 28	6 600	0.188	0.165	1.73
18	1921	Sept. 12	Sept. 25	14	3	3 to 5	3.71	3.21	Sept. 16	3.36	*U*	69	Sept. 16	Sept. 27	12	0.28	Sept. 16	3 380	0.075	0.265	1.30
19	1922	Apr. 5	Apr. 25	21	3	2 to 3	2.44	1.38	Apr. 9	2.25	*GM*	51	Apr. 9	Apr. 27	19	0.75	Apr. 18	7 340	0.307	0.116	1.67
20	1923	Mar. 11	Mar. 22	12	1	0 to 3	0.61	0.93	Mar. 15	1.00	*GL*	27	Mar. 23	Mar. 29	7	0.19	Mar. 25	4 160	0.311	0.051	1.39

NOTE.—*U* = greatest precipitation above Fort Dodge; *M* = greatest precipitation near Des Moines; *L* = greatest precipitation below Des Moines; and *G* = general distribution of precipitation.

* Winter storage and many light rains resulted in run-off exceeding rainfall.

TABLE 33.—(*Continued.*)

* Winter storage and many light rains resulted in run-off exceeding rainfall.

TABLE 33.—(*Continued.*)

Flood No.	Date of Storm. Year.	Time. Beginning.	Time. End.	Duration of storm, in days.	Number of stormy periods.	Number of days of more than 0.25 in. of rainfall.	Total effective rainfall, all days, with excess of 0.25 in.	Total rainfall, in 30 days before storm.	Date of maximum rainfall.	Maximum 24-hour rainfall.	Location of storm center.	Average temperature at Des Moines during storm, in degrees Fahrenheit.	Time of Flood. Beginning.	Time of Flood. End.	Duration of flood, in days.	Total run-off during flood, in inches.	Date of maximum flow.	Maximum 24-hour flow, in second-feet.	Ratio, run-off to rainfall.	Average rate of rainfall, in inches per day.	Ratio, maximum flow to average flow.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
Floods on Des Moines River Above Des Moines, Iowa—Period Before Drainage.																					
1	1903	May 8	June 26	50	6	10 to 20	9.14	3.17	May 25	3.70	*GM*	65	May 12	June 30	50	3.45	May 30	36 000	0.378	0.183	3.13
6	1905	May 9	June 7	30	4	5 to 8	6.54	2.06	May 10	2.20	*GU*	62	May 27†	June 9	..	0.51†	June 2	8 800	0.078†	0.218	1.44†
7	1905	June 8	June 21	14	2	2 to 4	1.82	6.32	June 10	3.49	*L*	71	June 10	June 21	12	0.21	June 10	6 750	0.115	0.130	2.33
8	1906	Feb. 11	Mar. 19	37	4	1 to 4	1.18	0.31	Feb. 13	1.45	*GL*(?)	28	Mar. 1†	Mar. 23	..	0.52†	.....	.....	0.440+	0.032	....
9†	1906	Mar. 22	May 4	44	5	1 to 8	3.44	1.08	May 1	2.00	*U*	50	Mar. 24	May 10	48	1.85	Mar. 30	13 700	0.538	0.078	2.14
Floods on Des Moines River Above Des Moines, Iowa—Period After Drainage.																					
10	1918	May 26	June 7	13	2	4 to 9	5.57	4.63	June 4	3.35	*MU*	70	May 30	June 17	19	1.65	June 7	41 000	0.296	0.428	2.85
11	1919	Mar. 9	Apr. 5	28	3	1 to 3	1.21	1.79	Mar. 15	2.44	*L*	45	Mar. 15	Apr. 7	24	0.89	Mar. 19-20	10 000	0.735	0.043	1.61
12	1919	Apr. 6	May 31	56	6	6 to 12	5.64	1.80	Apr. 7	2.10	*GL*	55	Apr. 8	May 31	54	2.83	Apr. 24	15 300	0.502	0.101	1.76
13	1919	June 1	July 18	48	8	7 to 13	8.06	1.94	July 4	3.10	*MU*	75	June 1	July 20	48	2.13	June 23	14 000	0.264	0.168	1.89
14	1920	Mar. 10	Apr. 12	34	8	2 to 7	2.24	0.81	Mar. 11	1.46	*L*	43	Mar. 12	Apr. 18	38	1.36	Mar. 14-15	12 000	0.607	0.066	2.00
15	1920	Apr. 14	June 7	55	10	3 to 11	5.74	3.01	May 11	2.20	*G*	58	Apr. 19	June 7	50	1.64	May 17	8 260	0.286	0.104	1.50
16	1920	June 27	July 27	31	5	5 to 10	6.01	1.86	July 6	2.30	*UL*	74	July 2	July 29	28	1.43	July 12	18 200	0.238	0.194	2.14
17	1921	May 16	June 22	38	5	5 to 11	5.73	2.71	May 26	2.79	*U*	75	May 21	June 22	33	1.02	May 30	9 430	0.178	0.151	1.81
18	1921	Sep. 12	Sept. 25	14	3	3 to 5	4.10	4.30	Sept. 16	3.36	*U*	69	Sept. 16	Sept. 30	15	0.56	Sept. 17	11 400	0.137	0.293	1.84
19	1922	Apr. 5	Apr. 25	21	3	1 to 4	2.32	1.50	Apr. 9	2.72	*GM*	51	Apr. 10	Apr. 27	18	0.83	Apr. 13-14	11 500	0.358	0.110	1.51
20	1923	Mar. 11	Mar. 22	12	1	0 to	0.84	1.06	Mar. 15	1.00*	*GL*	27	Mar. 24	Apr. 1	9	0.32	Mar. 28	8 000	0.381	0.070	1.33

† Incomplete record of flow.

TABLE 33.—(*Continued.*)

Flood No.	Date of Storm. Year.	Time. Beginning.	Time. End.	Duration of storm, in days.	Number of stormy periods.	Number of days of more than 0.25 in. of rainfall.	Total effective rainfall, all days, with excess of 0.25 in.	Total rainfall, in 30 days before storm.	Date of maximum rainfall.	Maximum 24-hour rainfall.	Location of storm center.	Average temperature at Des Moines during storm, in degrees Fahrenheit.	Time of Flood. Beginning.	Time of Flood. End.	Duration of flood, in days.	Total run-off during flood, in inches.	Date of maximum flow.	Maximum 24-hour flow, in second-feet.	Ratio, run-off to rainfall.	Average rate of rainfall, in inches per day.	Ratio, maximum flow to average flow.
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
Floods on Des Moines River Above Keosauqua, Iowa—Period Before Drainage.																					
1	1903	May 8	June 26	50	6	9 to 12	11.69	3.04	May 27	4.02	*G M*	65	May 29*	June 30	..	3.04*	June 1	97 140	1.26+*	0.234	2.82*
2	1904	Mar. 24	Mar. 25	2	1	1 to 2	0.54	0.90	Mar. 24	2.00	*G L*	36	Mar. 25	Mar. 28	4	0.16	Mar. 25	21 840	0.296	0.270	1.44
3	1904	Apr. 20	May 9	20	2	2 to 6	2.87	3.21	Apr. 25	3.09	*L*	57	Apr. 25	May 9	15	0.82	Apr. 26–27	43 800	0.286	0.144	2.11
4	1905	Feb. 23	Mar. 16	22	2	1 to 3	0.42	1.54	Feb. 24 / Mar. 10–12	1.10	*U*	37	Feb. 25	Mar. 18	22	0.73	Mar. 1	26 230	0.174 †	0.019	2.11
5	1905	Mar. 17	Apr. 11	26	4	2 to 5	2.20	0.76	Apr. 2	2.06	*G*	51	Mar. 19	Apr. 13	26	0.69	Mar. 25	16 530	0.313	0.085	1.65
6	1905	May 9	June 7	30	4	6 to 7	5.47	2.92	May 10	2.50	*G U*	62	May 11	June 9	30	1.18	May 15	23 360	0.329	0.182	1.59
7	1905	June 8	June 21	14	2	2 to 4	2.59	5.75	June 10	11.09	*L*	71	June 10	June 21	12	0.66	June 10	75 750	0.255	0.185	3.80
8	1906	Feb. 11	Mar. 19	35	4	2 to 6	1.40	0.40	Feb. 13	1.48	*G L*	28	Feb. 21	Mar. 25	33	0.91	Feb. 25	30 800	0.650	0.038	2.99
9	1906	Mar. 22	May 4	44	5	4 to 8	3.51	1.30	May 1	2.00	*U*	50	Mar. 26	May 10	46	1.79	Mar. 30	28 000	1.510	0.080	1.93
Floods on Des Moines River Above Keosauqua, Iowa—Period After Drainage.																					
10	1918	May 26	June 7	13	2	4 to 7	5.37	4.55	May 27	3.43	*M U*	70	May 28	June 18	22	1.21	June 10–11	39 300	0.225	0.413	1.90
11	1919	Mar. 9	Apr. 5	28	3	1 to 3	1.91	1.87	Mar. 15	2.44	*L*	45	Mar. 15	Apr. 8	25	1.07	Mar. 17	43 700	0.560	0.068	2.73
12	1919	Apr. 6	May 31	56	6	8 to 13	6.21	2.57	Apr. 9	2.13	*G L*	55	Apr. 9	May 31	53	2.60	May 8	41 800	0.418	0.111	2.27
13	1919	June 1	July 18	48	8	7 to 13	8.38	2.44	June 10	5.10	*M U*	75	June 1	July 20	50	2.16	June 8	35 900	0.258	0.175	2.23
14	1920	Mar. 10	Apr. 12	34	8	3 to 8	2.77	0.66	Mar. 24	2.07	*L*	43	Mar. 12	Apr. 18	38	1.70	Mar. 26	44 100	0.614	0.081	2.64
15	1920	Apr. 14	June 7	55	10	8 to 11	6.46	3.09	Apr. 19	3.34	*G*	58	Apr. 19	June 7	50	2.13	Apr. 21	33 300	0.330	0.117	2.10
16	1920	June 27	July 27	27	3	6 to 9	5.60	2.02	July 14	3.32	*U L*	74	July 2	July 27	26	1.01	July 14	32 200	0.180	0.207	2.03
17	1921	May 16	June 22	38	5	0 to 3	4.81	3.20	May 26	2.79	*U*	75	May 24	June 22	30	0.76	June 3	18 900	0.158	0.126	1.99
18	1921	Sept. 12	Sept. 25	14	3	5 to 6	3.89	4.93	Sept. 16	3.36	*U*	69	Sept. 16	Sept. 30	15	0.67	Sept. 21	26 900	0.172	0.278	1.60
19	1922	Apr. 5	Apr. 25	21	3	2 to 6	2.42	1.92	Apr. 9	2.72	*G M*	51	Apr. 8	Apr. 28	21	0.84	Apr. 17	23 500	0.347	0.115	1.57
20	1923	Mar. 11	Apr. 16	37	4	2 to 6	1.90	1.15	Mar. 11	1.51	*G L*	36	Mar. 23	Apr. 19	28	0.83	Mar. 28	26 200	0.437	0.051	2.86

* Incomplete record of flow. † Winter storage and light rains resulted in a run-off exceeding effective rainfall.

TABLE 34.—Summary of Floods on Iowa River at Iowa City, Iowa.

TABLE 34.—Summary of Floods on Iowa River at Iowa City, Iowa.

Flood No.	Date of Storm.			Duration of storm, in days.	Number of stormy periods.	Number of days of more than 0.25 in. rainfall.	Total effective rainfall, all days excess of 0.25 in.	Total rainfall in 30 days before storm.	Date of maximum rainfall.	Maximum 24-hour rainfall.	Location of storm center.	Average temperature during storm at Des Moines, in degrees Fahrenheit.	Time of Flood.		Duration of flood, in days.	Total run-off during flood, in inches.	Date of maximum flow.	Maximum 24-hour flow, in second-feet.	Ratio, run-off to rainfall.	Average rate of rainfall, in inches per day.	Ratio, maximum flow to average flow.
	Year.	Time.																			
		Beginning.	End.										Beginning.	End.							
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)
Period Before Drainage.																					
1	1903	May 8	June 26	50	6	9 to 12	8.30	3.04	May 25	3.63	*G*	65	June 1*	June 28	....	2.62*	June 3	21 500	0.316*	0.166	3.67
2	1904	Mar. 24	Apr. 1	9	2	1 to 2	1.24	1.38	Mar. 24	1.70	*G*	36	Mar. 25	Apr. 4	11	0.42	Mar. 25	8 800	0.339	0.138	2.67
3	1904	Apr. 20	May 9	20	2	2 to 6	3.08	2.77	Apr. 25	3.00	*L*	57	Apr. 23	May 12	20	0.66	Apr. 26	5 510	0.214	0.154	1.97
4	1905	Feb. 23	Mar. 16	22	2	1 to 3	0.82	1.38	Feb. 26	1.00	*G*	37	Feb. 23	Mar. 18	24	0.64	Mar. 12	4 700	0.783	0.037	2.14
5	1905	Mar. 17	Apr. 11	26	4	2 to 5	1.81	1.23	Mar. 19	1.75	*GM*	51	Mar. 19	Apr. 13	26	1.14	Mar. 25	6 500	0.630	0.070	1.76
6	1905	May 9	June 7	30	4	6 to 7	5.71	2.69	May 11	3.48	*GL*	62	May 11	June 9	30	1.91	May 21	8 540	0.335	0.190	1.58
7	1905	June 8	June 21	14	2	2 to 4	2.91	6.16	June 10	4.35	*L*	71	June 10	June 27	18	0.66	June 10	8 110	0.227	0.208	2.90
8	1906	Feb. 11	Mar. 19	37	4	2 to 6	1.37	0.84	Feb. 13	1.43	*GL*	28	Feb. 19	Mar. 25	35	2.05	Mar. 2	11 400	1.50 †	0.037	2.33
9	1906	Mar. 22	May 4	44	5	4 to 8	3.80	1.32	Mar. 26	1.37	*U*	50	Mar. 26	May 5	41	2.21	Apr. 3- 7	7 960	0.582	0.086	1.73
Period After Drainage.																					
10	1918	May 26	June 7	13	2	4 to 7	6.57	6.90	June 4	3.73	*UM*	70	May 26	June 19	26	4.22	June 8	35 300	0.643	0.505	2.58
11	1919	Mar. 9	Apr. 5	28	3	1 to 3	2.03	2.01	Mar. 16	1.55	*GM*	45	Mar. 10	Apr. 9	31	1.61	Mar. 16	8 820	0.783	0.072	2.00
12	1919	Apr. 6	May 31	56	6	8 to 13	7.20	2.52	Apr. 7	2.01	*GU*	55	Apr. 10	May 31	52	3.26	May 7	12 800	9.453	0.129	2.41
13	1919	June 1	July 18	38	7	7 to 13	7.32	2.84	June 17	3.27	*M*	75	June 1	July 18	48	2.09	June 6- 7	6 940	0.286	0.152	1.87
14	1920	Mar. 10	Apr. 12	34	8	3 to 8	3.25	0.65	Mar. 25	1.61	*G*	43	Mar. 6	Apr. 16	42	2.20	Mar. 17-18	8 130	0.677	0.096	1.85
15	1920	Apr. 14	June 7	55	10	8 to 11	6.31	3.04	May 12	2.29	*G*	58	Apr. 17	June 7	52	2.49	May 16	6 450	0.395	0.115	1.61
16	1920	June 27	July 27	31	5	6 to 9	4.86	2.11	July 5	2.63	*U*	74	July 2	July 27	26	0.57	July 8	3 550	0.117	0.157	1.97
17	1921	May 29	June 8	11	3	0 to 3	0.94	3.55	May 30	1.20	*G*	70	June 1	June 8	8	0.21	June 2	2 680	0.223	0.085	1.22
18	1921	Sept. 12	Sept. 25	14	3	5 to 6	5.47	5.27	Sept. 16	5.26	*M*	69	Sept. 16	Oct. 8	23	1.65	Sept. 23	14 300	0.302	0.390	2.35
19	1922	Apr. 5	Apr. 25	21	3	2 to 6	2.53	2.18	Apr. 9	1.36	*G*	51	Apr. 9	May 1	23	1.05	Apr. 11	5 780	0.415	0.120	1.48
20	1923	Mar. 11	Apr. 16	37	4	2 to 6	2.67	1.16	Mar. 15	1.27	*GL*	36	Mar. 27	Apr. 19	24	1.33	Apr. 7	8 420	0.498	0.072	1.79

NOTE.—*L* = near Iowa City; *M* = near Marshalltown; *U* = above Iowa Falls; and *G* = general. *Incomplete record of stream flow. †Winter storage and light rains resulted in a run-off in excess of rainfall.

The dark symbols indicate the run-off of the floods previous to drainage. In general, they fit in very well with the system of points indicating the behavior of the stream subsequent to drainage. If any difference exists there is some indication that storms concentrated on the upper portions of these water-sheds produced a larger run-off previous to drainage than at the present time. The correlation of rainfall and run-off in Fig. 35 is much better than that secured in Fig. 32, and would seem to be adequate to detect any significant difference which may have been caused by drainage. It is apparent from Fig. 35, however, that drainage has caused no measurable difference in the total flow of these streams during periods of high water and hence could not have been effective in producing higher stages in the Lower Mississippi River, only 4% of the water-shed of which has been subjected to drainage operations.

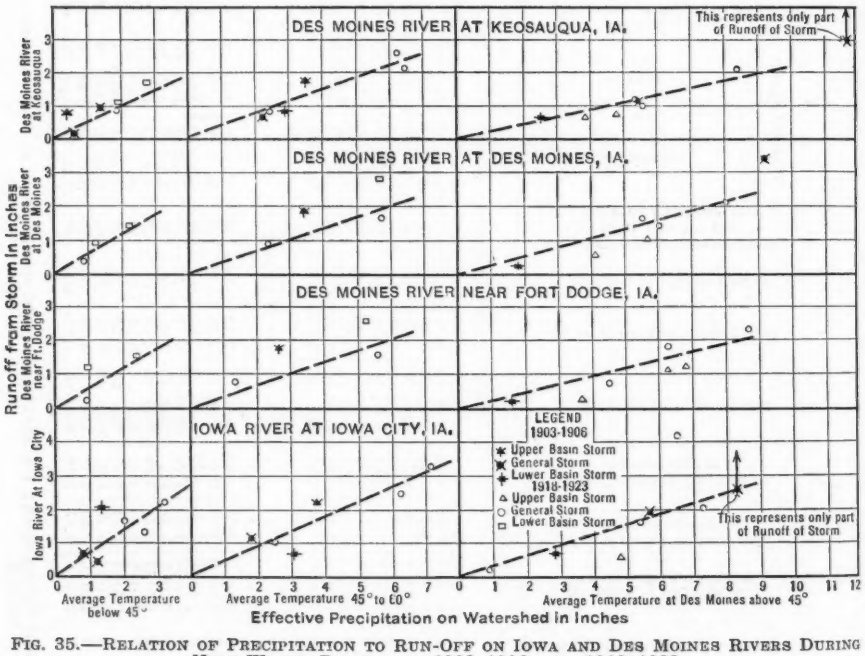
DES MOINES RIVER AT KEOSAUQUA, IA.
DES MOINES RIVER AT DES MOINES, IA.
DES MOINES RIVER NEAR FORT DODGE, IA.
IOWA RIVER AT IOWA CITY, IA.
This represents only part of Runoff of Storm
LEGEND
1903-1906
Upper Basin Storm
General Storm
Lower Basin Storm
1918-1923
Upper Basin Storm
General Storm
Lower Basin Storm
Runoff from Storm in Inches
Des Moines River at Keosauqua
Des Moines River at Des Moines
Des Moines River near Ft. Dodge
Iowa River at Iowa City
Average Temperature below 45°
Average Temperature 45° to 60°
Average Temperature at Des Moines above 45°
Effective Precipitation on Watershed in Inches

FIG. 35.—RELATION OF PRECIPITATION TO RUN-OFF ON IOWA AND DES MOINES RIVERS DURING HIGH-WATER PERIODS IN 1903–1906 AND 1918–1923.

Although no significant difference can be detected in the total flood run-off of these Iowa streams since drainage in contrast to that previous to drainage, the statement has often been made that certainly the peak run-off has been increased by these operations. It is difficult, indeed, to correlate the maximum flow of a storm with any single hydrologic factor, since it is dependent upon so many. Nevertheless, the maximum average 24-hour rate of discharge during these twenty periods of high water was plotted in Fig. 36 against the average daily precipitation during the storm which produced the run-off. Symbols similar to those in Fig. 35 were used, and lines were drawn from the point of zero rainfall and run-off through the open circles which represent

the peak run-off of those storms during 1918 to 1923 which were of general distribution or concentrated near the center of the water-shed. Again, it may be noted in Fig. 36 that storms the centers of which occurred in the lower part of the basin produced higher rates of run-off than the average, and those storms which occurred in the upper part of the basin (which had been drained) produced lower run-off than the average. The black symbols, representing the pre-drainage flood peaks, fit in very well with the general system of post-drainage points. If any difference at all exists, pre-drainage storms have produced higher peaks than post-drainage storms of the same intensity, although the storms are so few in number that this difference can hardly be considered to be of significance.

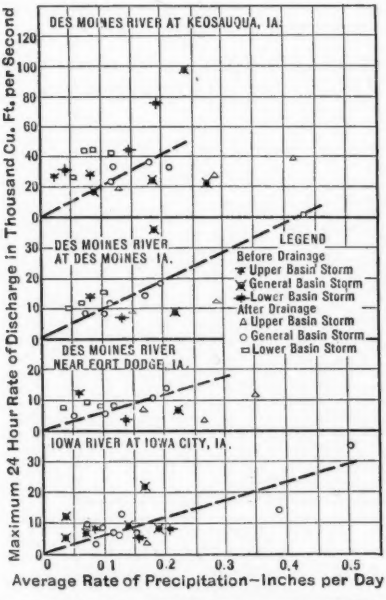
DES MOINES RIVER AT KEOSAUQUA, IA.
DES MOINES RIVER AT DES MOINES IA.
DES MOINES RIVER NEAR FORT DODGE IA.
IOWA RIVER AT IOWA CITY, IA.
LEGEND
Before Drainage
Upper Basin Storm
General Basin Storm
Lower Basin Storm
After Drainage
Upper Basin Storm
General Basin Storm
Lower Basin Storm
Maximum 24 Hour Rate of Discharge in Thousand Cu. Ft. per Second
Average Rate of Precipitation—Inches per Day

FIG. 36.—RELATION OF PRECIPITATION RATE TO MAXIMUM DISCHARGE OF DES MOINES AND IOWA RIVERS, 1903–06 AND 1918–23.

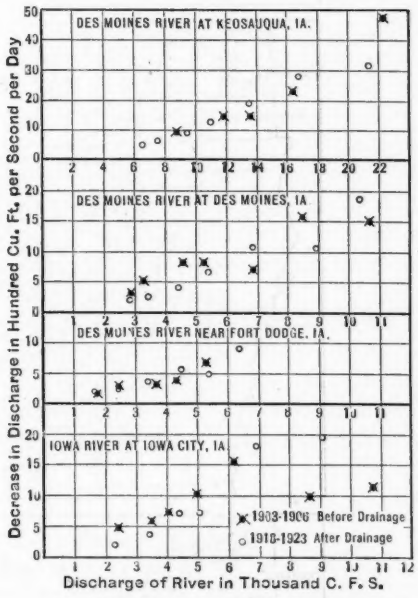
DES MOINES RIVER AT KEOSAUQUA, IA.
DES MOINES RIVER AT DES MOINES, IA
DES MOINES RIVER NEAR FORT DODGE, IA.
IOWA RIVER AT IOWA CITY, IA
1903-1906 Before Drainage
1918-1923 After Drainage
Decrease in Discharge in Hundred Cu. Ft. per Second per Day
Discharge of River in Thousand C. F. S.

FIG. 37.—RATE OF FLOW DEPLETION, DES MOINES AND IOWA RIVERS.

It remains only to describe the last and most sensitive test applied to the stream-flow records in this attempt to detect some appreciable change caused by agricultural drainage. This test was applied to the curve of diminishing flow following the peak of the storm, during those days of high water when there was no additional precipitation received on the basin. If within a given basin the surface storage has been decreased by drainage in such a fashion that the water is carried away with greater rapidity, the daily rate of run-off should decrease at a more rapid rate than that existing when the ground possesses a higher retentive value. The daily decrease in discharge at given stages of the stream, when undisturbed by subsequent precipitation, was averaged for all floods previous to drainage and compared with similar values subsequent to drainage. The results are shown in Fig. 37. It is natural that

FIG. 38.—DRAINAGE AREAS OF DES MOINES AND IOWA RIVERS, SHOWING AREA OF OPERATING DRAINAGE ENTERPRISES IN 1920.

the points should scatter somewhat, due to the difference in depletion rate caused by the location, distribution, and season of the storm, but it is significant that the black symbols representing the pre-drainage conditions show no appreciable difference from the open symbols representing the behavior of the stream subsequent to drainage. Thus, no appreciable difference in the retentive value of the water-shed in so far as it affects the stream flow is revealed by this analysis.

Certain storms which were of unusual intensity, and others which were concentrated over the land which experienced most of the drainage operations, may be of particular interest, and are shown in Figs. 31 and 38. Storms differ as much as human faces—there are no two alike in amount, location, and time of occurrence, hence no direct comparison is possible between particular storms in the periods before and after drainage. The storm occurring in May and June, 1903, is remarkable in the amount and general distribution of its precipitation, while the storm of May and June, 1918, produced an unusually high rate of rainfall over a large area within a few days.

This exhausts the writers' ingenuity in trying to find a way to detect a measurable effect of drainage upon flood run-off. It may be of interest, however, to note in conclusion that the largest known flood in the Des Moines River Valley occurred in 1851. The flood which reached a peak on June 1, 1903, almost equalled this flood in stage, but since 1903 no flood has occurred which has attained the height of these pre-drainage floods.

Likewise, the maximum known flood in the Iowa River Valley occurred in 1851, and again in July, 1881, a torrent only slightly smaller passed down the river. The flood occurring in June, 1918, in the post-drainage period, attained the highest stage which has been recorded since stream measurements were started, but it was exceeded in magnitude by both of the pre-drainage floods of 1881 and 1851.

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PAPERS AND DISCUSSIONS

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FLOOD CONTROL WITH SPECIAL REFERENCE TO THE MISSISSIPPI RIVER

A SYMPOSIUM

Discussion*

By Messrs. J. P. Snow, Frank B. Maltby, Morris Knowles, Charles H. Paul, and F. G. Jonah.

J. P. Snow,† M. Am. Soc. C. E. (by letter).‡—For at least seventy-five years the levees have been added to, strengthened, and perfected. This has resulted, of course, in compelling more and more of the flood waters to flow in the river channel. Before levees were built, any rise above bank level distributed itself over the surrounding territory, resulting in low gauge readings along the river. Records show that flood readings at Natchez, Miss., were:

1882	47.8	1913	52.6
1892	48.1	1916	53.6
1897	49.8	1922	55.5
1903	50.4	1927	56.6
1912	51.4		

These were maximum floods for the past fifty years. The increases in height were due, without doubt, quite largely to the continual improvement in the effectiveness of the levees. The 1927 flood, in spite of the vast volume of water lost through the various breaks, was so high at New Orleans that an artificial crevasse, involving large reparation payments for damages, had to be made to save the city. Further efforts to control maximum floods by levees

* This discussion (of the Symposium on Flood Control with Special Reference to the Mississippi River, presented at the Fall Meeting of the Society and the meeting of the Waterways Division on October 12 and 13, 1927, at Columbus, Ohio, and published in December, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., Boston, Mass.

‡ Received by the Secretary, October 14, 1927.

alone would seem to be absurd. In 1927, if there had been no breaks above Baton Rouge, thereby forcing all the great flood to follow the river channel, the levee system of its lower course would have looked puny indeed.

The normal flow must be held in the present channel of the river to serve the cities and landings along its banks. The surplus of flood flow, above what can be safely carried by the present levees, perfected to a consistent standard, should be taken off by adequate spillways at strategic points. These points are naturally at the mouths of large tributaries, where a great influx of water seeks to enter the river.

The first large tributary north of the Gulf is the Red River, with its branch, the Black River. This is the natural place for the principal spillway. A large basin should be provided there, making use of old bayous, with a substantial spillway with stop-logs along its southern border. The crest of the spillway should be above the level required for satisfactory navigation in the river during the summer season and below the gauge reading at which the present levees, after repairs, can carry the flow safely to the sea; perhaps but little above what is known as bank-full for the general run of the river. It should be long enough to prevent a flood from rising so high as to seriously endanger the levees of the lower river. Perhaps a height of 3 ft. below the tops of the levees may be assumed as a proper summit of flood from which to compute the length of the spillway. It should be designed to carry, at maximum flood, two-thirds of the water at this point seeking transportation to the Gulf, leaving one-third for the main river to handle.

The spillway must be built as a mere overflow without abrupt fall on the down-stream side. Undercutting at the toe of the masonry must be prevented by sheet-piling properly protected against decay. The Atchafalaya Channel must be cleared to carry the flow from the spillway without obstruction. Width in its upper course is necessary, rather than depth. The slope from the spillway to Grand Lake will be much steeper than that of the main river, and protection against undue scour must be provided. This will be particularly needed for a distance below the spillway. No abrupt fall should be allowed. A particularly wide channel is advisable between Grand Lake and the Gulf, so that the lake will be practically an arm of the sea.

If levees are needed along the Atchafalaya it may be best to construct drainage canals to take the tributaries on each side to Grand Lake rather than to build levees along these side streams. The work of construction is simple in detail throughout, but its magnitude will call for a great expenditure. Its probable ultimate benefit, however, justifies its cost.

This by-pass should hold the flow at a level that will make the river safe as far north as Vicksburg, Miss., at least, and it should prevent overflow of the Red and Black Rivers if their levees are in order. It would seem justifiable to ascertain what the by-pass would do in a flood toward ameliorating the conditions in the upper section of the flood area before attempting further work. If necessary, another spillway could be constructed near the mouth of the Arkansas, or possibly opposite Greenville, Miss., and a channel cut *via* various bayous and creeks into Black River and thence to the Red River Basin. This

spillway would probably lower the main river enough, so that the St. Francis Valley would be made safe, and danger would be prevented on the east side at Mound Landing and other points.

The objective is to run off the top of the flood, so as to render the present levee system safe for the main river below Cairo, Ill. Hydraulic data collected during the last seventy-five years should be sufficient to enable computations to be made for the safe design of the structures contemplated herein.

FRANK B. MALTBY,* M. AM. SOC. C. E.—The problem of flood control as a whole is discussed in this Symposium. Other problems treated are the effect of rainfall and run-off on the stage and discharge of the river, forestation, reservoirs, levees, and spillways and auxiliary channels as means of controlling the floods, so as to prevent a repetition of the disastrous results of the record flood of 1927.

All the papers have been prepared by men of experience, of close association with the existing conditions, and with access to an enormous amount of data that have been accumulated by the Mississippi River Commission and others. The sole object of this wide discussion was not simply to add to the available fund of engineering information, nor for the benefit of engineers only. The whole country is vitally interested in the problem, and is looking to the Engineering Profession for a solution.

Engineers are seldom unanimous in their opinion on any subject, and there has been a wide divergence in the ideas expressed on this one, but from discussions of this kind it should be possible to arrive at a consensus of opinion of value to those responsible for the enactment of necessary legislation.

It is fortunate that, thus far, prominent engineers have led in the discussion of this problem as it has been presented to the public. Especial reference is made to the address of General Jadwin delivered at the Flood Control Congress at Chicago, Ill., in June, 1927. It has so often happened that such conventions have been influenced by poor advice, and have adopted unwise resolutions that have had considerable weight with the uninformed. Engineers have often been content to sit back, shrug their shoulders, and remark that the whole scheme is impracticable. How much better and more valuable it is to lead and to guide than to follow and to criticize!

When it is remembered that the Mississippi River, below the mouth of the Red River, drains from 40 to 50% of the entire area of the United States, and that the alluvial valley of the river contains about 19 000 000 acres of fertile land subject to overflow if unprotected, the assumption that the problem of flood control on the Mississippi is a National one, is well warranted. The whole matter assumes such large proportion and affects such large areas that the solution and the control can only be handled by the Federal Government.

Viewing all the vast and extensive work done by the Mississippi River Commission during the last forty years, it should be particularly remembered that not the least valuable are the accurate, detailed surveys and maps that

* Cons. Engr., Norristown, Pa.

have been made, the establishment and maintenance of numerous gauges and records, the extensive discharge measurements made at various stages, sedimentation studies, and the collecting and recording of all kinds of data concerning the river. The speaker doubts if there exists anywhere in the world any more complete and accurate information of any river than that collected by the Mississippi River Commission. The voluminous records have made possible the scientific determination of the various features of locating and building levees, bank protection, dredging, etc.

The speaker wishes to advocate the Commission plan for the work of such magnitude as is being done on the Mississippi River, and to make a plea for the continuance of such a form of control of the work to be undertaken by the Federal Government. It is believed that a mixed commission of seven members, as well chosen as the members of the Mississippi River Commission have been thus far, can handle a problem of such magnitude much better than it can be done by any other form of control. Every one appreciates the advantages of centralized control; but this is too big a project for any one man even with the assistance he might command. The membership of the Commission may change, and it does; but all the members do not change at the same time. This results in a fixed and continuing policy which is not affected by each change of administration.

The economic problem of the Mississippi River covers various features, that of providing for the needs of navigation at low stages, prevention of erosion of the banks, as well as control of the flood waters, etc. This speaker will confine his discussion to the latter feature. Floods are caused by rainfall and run-off, as has already been discussed.* No man has any control whatever over the rainfall and its distribution, and, consequently, engineering efforts can only be directed to a proper and reasonable control of the floods after they have reached the river. The only control necessary or desirable is some method of preventing the loss of life, overflow of valuable and fertile lands, railroads, highways, cities, and towns.

In speaking of the control of floods after they have reached the Mississippi River, it is proposed to include that part of the river below Cairo, Ill., or its vicinity only. This limitation will not meet the approval of engineers who feel that the flood solution lies in the construction of reservoirs near the head-waters of the streams. It may be that the construction of reservoirs on the Allegheny, Monongahela, Miami, Upper Mississippi, and other rivers, may be warranted for local control, but it is believed that it has been conclusively demonstrated that any flood control in the Mississippi River delta by reservoirs would necessitate their construction in the delta itself, or immediately adjacent to it. Such reservoirs would require such enormous areas to be condemned and abandoned as to make it impracticable. The problem is to save from overflow the 19 000 000 acres in the delta, and the States can not give up 4 000 000 or 5 000 000 acres for reservoirs, if it can be done in some other way.

* *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, pp. 2454 to 2615, inclusive.

The papers* on spillways, auxiliary channels, and their various features as means of flood control are most valuable and interesting. The methods outlined may assist to some extent, but the speaker is firmly convinced that the real solution lies in the construction of adequate levees properly located. Engineers are familiar with the history and growth of the levee system. Their construction has been going on for many years, and the acreage of land protected has steadily increased with the growth of the system. To the present time practically the only method used for controlling the floods, or protecting the land from overflow, was that of building levees.

When the disastrous flood of 1927 came, a cry went up that the levee system had failed, and that some other solution of the problem must be found. The levees have not failed but, on the other hand, inadequate as some of them were in size, they did, in 1927, protect one-third of the area subject to overflow, and the protected area contains a much larger percentage of cultivated land than that flooded.

One might as well contend that, because costly roads have failed under the increasing demands of automobile traffic, the public should seek some other means of transportation. Instead of abandoning the automobile, people build better roads; so instead of abandoning the levee system, better levees must be built.

It is interesting to note that only one break occurred in Government levees built to standard height and cross-section based on the flood of 1912-13. It does not seem that the engineers made any very great error in designing the levee system as proposed. It is true that occasionally a fire-proof building burns, or a bridge or a dam fails, but that is no reason why every one should stop building fire-proof buildings, or dams, or bridges. Build them better and stronger; also, build better levees if necessary.

The greater part of the area inundated by the failure of levees was caused by breaks in those built by State or local authorities, and largely on the tributaries, where they were not built to the standard cross-section. This statement is not made in criticism of the local authorities or their engineers as they did the best they could with the money available. It only emphasizes the fact that the jurisdiction of the Mississippi River Commission should be extended to embrace the construction of levees on the tributaries as far at least as the protection of the Mississippi Delta demands.

There is no desire on the part of the speaker to enter into a discussion of the size, height, or cross-section of an adequate levee. These are details, which can safely be left to those in responsible charge of their construction. It is conceded that provision for a flood, even one somewhat higher than that of 1927, should be made, and that the present standard cross-section is somewhat too small, as in some places the line of saturation falls outside the base on the land side. This results in greatly increasing the cost of maintenance, because a leaking levee must be watched continually as it is liable to fail at any time. The section should be such that in any given material the line of saturation should be always within the base, with water against the levees to within 1 ft., at most, of the top.

* *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2532 *et seq.*

The speaker does not wish to be understood as advocating levees, and levees only, as the entire solution of the problem of flood control on the Mississippi River. In determining the grade line of the top of the levees, due consideration should be given to the economy of construction. It would seem hardly necessary to build levees that will provide for the extreme floods that may possibly occur once in 50 or 100 years.

For such unusual floods it may be, and probably will be, more economical in cost, and provide greater safety, to build controlled spillways that will take care of the floods above a predetermined elevation. The time of arrival and height of the crest of the flood will be known and can be provided for by the manipulation of the spillways. Instead of spillways with mechanical control, those of the automatic siphon type might be used. The latter type has proven very satisfactory in operation on a smaller scale, and is worthy of very careful consideration. In any event they should be of large and ample dimensions and of sufficient size to prevent the flood in the channel between the levees going above a predetermined height, even with a discharge as great as may be thought to be possible. A comparatively small acreage of land would be flooded by these spillways and that for a short time.

Owing to the universal use of automobiles and trucks and the scarcity of boats, it would seem desirable to construct a highway on top of the levees to permit ready access during high water for repairs and inspection. Maintenance of the roads would result in more careful inspection of the levees.

The only feature of this discussion that the speaker wishes to emphasize is that of levees. It is believed that they furnish the only adequate method of controlling the floods of the Mississippi. It may be that they can be supplemented to a certain extent by spillways as mentioned, but the main reliance must be found in the levee system.

Remember that the levees protected 6 000 000 acres of fertile land during the recent unprecedented flood. The work done and the money expended on levees to date is not wasted, but will be utilized as a part of the system to be completed to the dimensions that experience has found necessary. Control by other methods, at the best, is problematical. Engineers know the weak features of the levee system and can correct them.

Morris Knowles,* M. Am. Soc. C. E.—The recollections of the recent disaster in the Lower Mississippi River Valley are still fresh in every one's memory. The Ohio River is an important tributary of the Mississippi and it frequently contributes disasters to the lower river. It also has problems of its own, some of which are of local character and have already been solved, notably at Dayton and Columbus, where one can see the works of construction resulting from thorough study.

The head-waters of the Ohio River are at Pittsburgh, Pa., where the Allegheny and Monongahela Rivers unite. At this place, there still remains unsolved (so far as construction is concerned) and lacking in determinative opinion, except from study, "the most important [flood problem] in the United States, in which a single city is the chief interest concerned and in which the

* Pres. and Chf. Engr., Morris Knowles, Inc., Pittsburgh, Pa.

chief reliance for protection so far proposed is an extensive reservoir system".*

First impressions of the papers and discussions in this Symposium indicate a note of discouragement. It has been stated that if the present levees had held, the recent flood would have been 6 to 8 ft. higher at several places between Helena, Ark., and New Orleans, La. The Mississippi River Commission reports that a discharge 25% greater may be expected as a probability and that to care for such a flood, within "levees only" would require an increase in present levee heights of from 7 to 20 ft., in the stretch of river to 900 miles below Cairo, Ill.

It is further stated†:

"Any project which contemplates increasing the heights of existing levees at certain localities about 15 ft., not only affects the item of cost, but raises the question of the practicability and feasibility of maintaining such a levee line after it has been constructed."

* * * * * * * * *

"* * * no matter how much money is expended on the levee line, it will still be at the mercy of the combination of a muskrat, a dark night and a careless levee inspector. * * * To try to construct a levee line which will resist the river flow under all the imaginary conditions which now fill the public mind, is to attempt the impossible".

Again, it is said‡ that:

"* * * an 1 800-mile levee line of the height necessary to contain such a flood, would carry with it a menace to the people living behind it that should be avoided if practicable".

At the same time, few, if any, words of encouragement seem to be forthcoming about any of the alternative methods of substitution or means of alleviating conditions and lessening the volume of flood waters to be restrained by such a levee line. Certainly, if these are the facts, then the people of the Lower Mississippi Valley should be informed frankly and told that nothing will avail against the greater catastrophe that may and probably will happen. Common honesty and fairness dictate that they should not be lulled into a sense of false security.

The speaker believes, however, that all is not as dark as it is colored, and that the knowledge, training, ingenuity, and research of the American engineer will not falter at the problem. Granted the first three, then research must be brought into action. Mr. Morgan§ has well argued, based on long and familiar experience with such problems, for the efficacy and necessity of information, resulting from long-time study of an accurate and detailed nature. As to the opportunity for this, as distinguished from the need of haste, people are assured that by 1928 the line of levees will be re-established to the full extent at which they existed prior to the flood of 1927. It has been stated‖ that:

* "Flood Control," by the late H. M. Chittenden, M. Am. Soc. C. E., *Transactions*, Inter. Eng. Cong., San Francisco, 1915 (Waterways and Irrigation), Paper No. 30, p. 110.

† *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, pp. 2460 and 2469.

‡ *Loc. cit.*, p. 2509.

§ *Loc. cit.*, p. 2532.

‖ *Engineering News-Record*, October 20, 1927, p. 636.

"Our lines of defense will be closed before the next flood season. * * * Although not entirely completed to standard grade and cross-section, it would seem that, by all the laws of probability, we should be reasonably safe until the revised project can be started."

If this is true, then why all the haste to spend fabulous millions until sufficient data have been received on which to base such large expenditures? American engineers have a real duty to educate the public so as to prevent a stampede of Congress on this important question, to the end that no unwise conclusions be reached, under the false urge of haste, which may burden the country with an erroneous policy for generations to come.

Pittsburgh Problem.—The speaker has been asked to discuss the Upper Ohio situation. This problem and the proposed solution, the reasons for delay, the progress to date, and the possible effect on the lower river have all been subjects of misconception and misunderstanding.

At times, the flood flows from the Ohio River Basin may be an important contribution to great disasters on the Lower Mississippi. While the drainage area is only one-third that of the Missouri (the great tributary of the Mississippi), the rainfall is normally about the same over this area as upon that of the greater river. It frequently contributes one-half the flow to great floods in the lower river. It is possible, also, for its outpouring to synchronize with that of the lower river.*

Western Pennsylvania, including the Allegheny and Monongahela River Basins and the upper portion of the Ohio, together with their tributaries, have been visited by disastrous floods for half a century, culminating in three such events, in 1907 and 1908. The flood of 1907 is the greatest one of which there is any record. It is possible, however, by certain combination of circumstances, distribution of rainfall, degree of saturation of the soil, and conditions of the surface, to have a flood at Pittsburgh at least 10 ft. higher than that which was so disastrous in 1907.

In twenty years, including 1908, the damages at Pittsburgh have been estimated to amount to $17 000 000 of which $6 500 000 was for the three floods of 1907 and 1908. In 1907 the total damage in the Ohio Valley was about $100 000 000. While the streams are reasonably well constricted and the downtown district of Pittsburgh, on both banks of both rivers, contains only about 3 000 acres, about 1 600 acres are affected by large floods. This area includes 100 office buildings, 17 miles of main-line steam railroad, 33 miles of streets, and 7 bridges rendered inaccessible.

Effect of Encroachments.—Statements are constantly made, even in professional circles, but sometimes without adequate basis, that man's encroachment has been an active agent in increasing, intensifying, and even in producing recent floods. Care should be exercised in such discussions to differentiate between encroachments by embankments in wide alluvial valleys (as they have been described), and those in narrow stretches, restrained by comparatively high elevations on either side, such as exist in many of the upper tributaries. It is, of course, true that floods are more disastrous in recent years, but to a large extent this may be because of the greater monetary value of improve-

* *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2484.

ments within flood areas, due to the extension of man's activity. In some cases, of course, encroachments do intensify floods above such a situation. However, where such restriction is made, it is often possible—in rectifying, straightening, and narrowing the streams—to make the flood discharge of the same capacity and sometimes even greater, by reveting, smoothing, and straightening the banks. Thus, the advent of human occupancy may, in some instances, and can, in most situations, be made not to intensify but to lessen flood troubles.

Organization of Pittsburgh Flood Commission.—Realizing the adverse effect of these occasional visitations and, for the purpose of studying the problem, the business interests of Pittsburgh (acting on suggestions of local engineers through the Chamber of Commerce), created what was called the Pittsburgh Flood Commission, in 1908. This commission included representatives of Allegheny County, the City of Pittsburgh, manufacturing and industrial companies, and the transportation interests. There was a goodly representation of engineers and these were grouped and designated as the Engineering Committee.*

In addition to the question of securing accurate information (by surveys and otherwise) upon which to base an intelligent consideration, the Commission devoted considerable time to studying the effect, both physically and financially, as to the floods which had occurred from time to time. It may be said frankly that the problem was approached, in the first instance (as such problems usually are), with a desire to relieve the local situation. It was thought that the raising of the street and land levels near the rivers, with the possible building of walls, or other barriers, along the water-front, would be of immense benefit. As a result of this discussion, in the subsequent year of 1915, about $500 000 was spent on such regrading work by the City, public utilities, and private owners. Many streets in the low areas of the city were raised and re-arranged to considerable advantage to their use for business purposes. The occasions and the extent of flooding were diminished thereby.

Local Solution not Sufficient.—However, it soon appeared that it was quite possible that such local or selfish consideration would not solve the problem as completely as it should be undertaken. Such restraining walls might have slight effect on some other places near and above the city, but would not materially lessen floods below the city. In addition, a little consideration showed that, at Pittsburgh, as at many other places, while there is too much water at certain seasons of the year, there is too little at other times. This condition sometimes renders the banks of the river and the bed unsightly, diminishes the navigable capacity at times of low stream flow, and increases the injurious effects of pollution and contaminating influences. Dilution has been found to

* There were nine engineers, seven of whom were members of this Society, three of whom, Messrs. Swenssen, Wilkins, and Long, have since died. The members of the Engineering Committee were: E. K. Morse, M. Am. Soc. C. E., Chairman, the late Emil Swenssen, M. Am. Soc. C. E., Vice Chairman, the late William G. Wilkins, M. Am. Soc. C. E., Vice-Chairman, George S. Davison, Past-President, Am. Soc. C. E., Paul Didier, M. Am. Soc. C. E., the late S. C. Long, Gen. Mgr., P. R. R., Julian Kennedy, Morris Knowles, M. Am. Soc. C. E., George M. Lehman, M. Am. Soc. C. E., Secretary and Executive Engineer, and A. B. Shepherd (resigned), Superintendent, Pittsburgh Bureau of Water.

be an important factor in the power of bodies of water to take up and render foreign substances innocuous.

As a natural result of such realization, the Pittsburgh Flood Commission early began to study the possibilities of lessening the large volume of flood waters which comes down to the city at certain times, and increasing the flow during low periods. This naturally meant a study of the drainage area, which is about 19 000 sq. miles, of which, 11 580 sq. miles is on the Allegheny and 7 340, on the Monongahela; and 20% of this area is within Pennsylvania; it also comprises about 10% of the Ohio drainage area tributary at Cairo.

Reservoir Possibilities.—By careful surveys and studies of the area within the water-shed of the Allegheny and Monongahela Rivers, it was found there were forty-three possible reservoir sites, which were considered practicable and worthy of intensive investigation. More could have been found if such study had been conducted a half century ago or even a generation prior to this. It is possible that not all of these, found in 1908 and 1909, will be considered now as feasible sites. Thus, does Man by his own works and extensions, sometimes render the problem of conservation more difficult.

Careful and selective study of the availability, effectiveness, and efficiency of the various suggested reservoirs revealed that, of these forty-three, seventeen would be practically and sufficiently effective to take care of all the floods which had ever been experienced at Pittsburgh, except that of 1907. This flood would have been reduced about 8 ft., the average reduction being 10 ft. for all floods. Additional important considerations are, that the use of such reservoirs would not only have reduced flood stages, but increased the low-water flow to a measurable extent; the Allegheny low-water discharge would be increased three times, and that of the Monongahela six times.

Each flood in the last twenty-five years has been analyzed as to its time of occurrence, duration, and intensity, and the effect that would be produced by each of the seventeen reservoirs proposed on the two water-sheds. Of course, there is no guaranty that floods would occur in the same situations, to the same extent, or with the same periodicity as to extent of time or intensity. However, that which has occurred through several years of recorded history is quite persuasive as to the probable climatological, meteorological, and physical conditions that are likely to occur in the future. Periods of high water in the streams of the western slope of the Alleghenies occur more frequently at certain times of the year, just as it is more often that there is low water in the autumn. Of the floods in the Pittsburgh District, 75% of which there is any record, has occurred between December 1 and April 1, and there never has been a flood of any marked intensity during the months of September, October, and November.

Effect on Lower Rivers.—Some ill-considered comments have been expressed as to the probable effect or non-effect of such reservoirs, wherever they may be placed, on the flow of the Lower Mississippi River. The Pittsburgh Flood Commission and its engineers have never made any claims as to such results, because they have not investigated and studied the effect of the reservoirs on the Allegheny and Monongahela Rivers, on the Mississippi floods.

They have, however, made some study extending down the Ohio River about 90 miles. The greatest flood on record at Wheeling, W. Va., would be reduced, by this proposed system of reservoirs, from 13 to 14 ft., and the low-water flow would be increased three times its minimum, on the basis of average normal assumptions as to condition of reservoirs.

Below this point on the Ohio River, there are many large tributaries, and it would be idle, with present lack of data, to say what the effect of the upper reservoirs would be on the high and low-water stages of the river below, to Cairo. It is undoubtedly true, however, that for some distance below Wheeling, perhaps even extending to Cincinnati, Ohio, there would be a marked beneficial effect by the construction of the seventeen reservoirs. The most important thing, however, is that the solution proposed above Pittsburgh, be studied in connection with the topography, possible locations, distribution of rainfall, and stream flow on the important tributaries below Wheeling, so that idle and immature conjecture as to effect will be replaced with sound opinion founded on careful estimate. It may then appear that favorable sites are found, with possible reduction of flood heights and increase of low-water flow, as a result of the use of such reservoirs. If such be found then, in order to obviate possible injurious interference or clashing, in the use of one reservoir system on another, it points to the need of National control, as to operation and manipulation of reservoir outlets.

National Character of Problem.—It is a National question, because that which is done in one part of the country may vitally affect the lives and property of people in another part. This does not necessarily mean Federal appropriation and expenditures alone, for an important National policy is, to create that feeling of responsibility as well as action, to the end that payment for benefits shall be assessed upon and paid to some degree by those particularly receiving benefits. It is well to have, however, the conception of National responsibility, as well as unified control, in order that such a system of reservoirs may be basin-wide in influence. The manipulation of the large systems of storage reservoirs on water supply projects indicates too the kind of centralized control which would make such a system effective and helpful rather than inefficient and hurtful. Perhaps it should be pointed out that the criterion is not the storage of the entire volume of flood flow, but the retardation of that portion of the excess which, added to the flood flow of other tributaries, produces the disastrous result.

It is recognized, in speaking of possibilities on the Ohio River water-shed, that this has been a matter of controversy for several years. Both proponents and opponents have been outspoken in their assertions, beginning with the monograph by Charles Ellet, Jr., in 1853. However, since then, such discussions have related to the usefulness of reservoirs to increase low-water stages for navigation, in preference to the use of locks and dams for such purpose. Now that canalization is about completed, this question need not be considered and engineers can discuss reservoirs, if and when they have adequate data as to their usefulness in restraining floods. It remains, however, to determine the actual facts as a result of careful and conservative

surveys. It is hoped that the forthcoming reports, requested by the President and Secretary of War, as a result of the Mississippi disaster, will throw some light on such agencies and the best location, construction, and effective use of reservoirs in conjunction with that other very valuable agency of restraining the lower river within suitable and safe levees. It is suggested, however, with present meager information as to topographic maps, stream flow, and precipitation at salient points all over the country, that nine months is too short a time to conclude a study on such a project adequately and comprehensively. If stations for measuring rainfall and stream flow were distributed over the country and universal topographical maps existed, it now would be possible to solve these problems much more quickly.

No Conflict with Power Uses.—One frequently hears the axiomatic statement that empty reservoirs are needed for flood protection and full reservoirs for navigation and water power. In these days when considerable attention is being paid to the development of water resources for power, that is, "white coal", one is constrained to look into the question, is there any necessary antagonism? The Pittsburgh Flood Commission and its engineers believe there is not. It does not, however, advocate the building of great hydroelectric plants in connection with the construction of reservoirs for flood protection, unless real economy and efficiency are to be obtained thereby. The need and request for flood protection should not be used as a guise for securing public appropriations for the development of water power not yet needed; but it may be prudent, in arranging for design, that future possibilities of power requirements be not forgotten.

However, although stated head and constant supply of water are needed for power, and although empty reservoirs are the safe criterion for flood protection, it is still asserted that both uses are not incompatible. By this is meant use at a location and site and not as to volume or storage within certain elevations. The explanation is that a certain volume is needed for flood storage, and a certain head is needed for power. There is nothing antagonistic in providing necessary flood storage at that upper portion of the dam and within the upper reaches of the reservoir above the elevation required for power. It will frequently be found true that the building of the dam, at a slight increased height necessary for flood storage, on top of a dam required for power, will be less expensive than the building of a separate dam, providing the same amount of storage solely for flood protection.

This point is emphasized, because of the misconception previously mentioned and in order that the position of the Pittsburgh Flood Commission may be made plain; that is, it is not opposed to the development of power, where economically wise, on the streams which unite at Pittsburgh. It has always taken the position of welcoming the investment of private capital, where it can be found to be efficiently done, in the production of electric power by the use of flowing water. It is realized that, even if there cannot be an arrangement to build a dam at increased height, there is a valuable retarding effect, due to flow over the crest of a dam as a weir, thus raising the pond level, whenever there is a heavy precipitation or run-off.

As an example of recent investigation into this subject, the Flood Commission made a study on three proposed sites on the Clarion River, one of the main tributaries of the Allegheny, having a drainage area of 1 260 sq. miles. The total storage proposed for power purposes was 32 000 000 000 cu. ft. It was even in excess, by more than 20 000 000 000, of storage proposed at some similar sites by the Flood Commission for the complete control of the Clarion River. However, such power dams, not built primarily for flood purposes, would not have as complete effect as if those proposed by the Flood Commission were to be erected. However, they would have had an important deterring and retarding effect on most of the flood heights at Pittsburgh during the last quarter of a century. Of almost equal importance was the fact that the constant flow of water from these power dams would increase the low-water flow of the Allegheny River about three times, and this from the development of reservoirs on one tributary only.

Why the Delay?—It is natural now to ask, if this project be so meritorious, why the country has not heard more about it and something of its construction. The answer is plain. The Allegheny River has portions of its water-shed in two States, New York and Pennsylvania. The Monongahela River has its water-shed in three States, Maryland, West Virginia, and Pennsylvania. Reservoir sites have been proposed in several States and affect the flow of the stream in each. It is not possible, therefore, for the city, county, or even the State of Pennsylvania to carry out the project in its entirety, or in the comprehensive way in which it should be undertaken. Such a contemplation should not be undertaken in a small way—it means National action and control. Heretofore, and from time immemorial, the consideration of the Federal Government has related to the use of large streams for navigation only. The question has been how much would the destruction of the forest, the building of a bridge, encroachment on a stream, or the building of a reservoir affect navigation. If seriously and injuriously, then it became a matter of interest to National agencies and, if this were not true, then Congress was not concerned.

Recently, however, there has been a change of consciousness. Perhaps now, with a realization of the recent calamity of the Mississippi River disaster, with an appreciation that the drainage area, population, and wealth comprehends about 40% of the Continental United States, for the first time, Congress may have an appreciation that flood control and regulation of streams is a question to be considered within the whole drainage area, from the source to the mouth, and is a problem for National study, construction, and control.

Forest Benefits.—One of the first acceptances of this view happened when the Week's Appalachian Forest Bill was passed by Congress, in 1911, as a result of which many thousands of acres of land have been secured on the Appalachian Mountain Range for the purpose of protecting slopes from erosion and growing forest cover for its many beneficial purposes. The Pittsburgh Flood Commission early joined hands with the National Forest Commission and upheld its work for this purpose. More than $15 000 000 has been spent in Eastern States, of which $1 200 000 has been on the Allegheny

and Monongahela water-sheds, acquiring 367 000 acres of land. Ultimately, it is expected that there will be secured more than 860 000 acres in West Virginia, 80 000 acres in Maryland, and 425 000 acres in Northwestern Pennsylvania. Some of the proposed reservoirs are on lands contemplated for National Forest procurement.

Authoritative Review.—It is appreciated that National action cannot come without Congressional approval, based on a report from authoritative governmental agencies. Shortly after the report of the Pittsburgh Flood Commission was issued, it was reviewed by the U. S. Engineer Office, having to do with the question of the effect of the proposed reservoir system on the navigability of the Ohio River. No question was suggested of course that it would in any way supplant, or replace, or do away with the construction of the locks and dams which are now nearing completion. However, it was realized that at several times in years of low flow, there has not been sufficient water in the Monongahela to allow the boats to pass through the locks without letting out water from the upper pools of the river formed by the dams. Such was the rather general experience during the drouth of 1908.

It was found by the Government engineers that, while such a system of reservoirs would be feasible and practicable, the effect of navigation would be little and would not warrant a participation in expense in excess of 10 per cent. More recently, however, with the newer view of National responsibility as to control of streams, another survey and investigation is in progress by the Federal authorities.

This was brought about through the joint action of the State of Pennsylvania and the National Congress. The State of Pennsylvania, at the session of 1919, appropriated $25 000 to the National Government for making a study of reservoirs proposed by the Pittsburgh Flood Commission and their general effect on the flood question. The Chief of Engineers, under date of December 21, 1916, recommended that the War Department be authorized to make such study. On May 31, 1924, a similar bill was passed by Congress, appropriating $25 000 for this purpose, to be used with the $25 000 from the State of Pennsylvania.

As a result, a study has been going on since 1925, under the District Engineer Officer at Pittsburgh, to look into the question of the feasibility and advisability of reservoirs on the Allegheny and Monongahela drainage areas, to control floods, to increase the low-water flow, and the attendant effect on navigation. This examination is most thorough and extended. It is to be expected that study and criticisms of the location and method of arrangement of the reservoirs proposed by the Pittsburgh Flood Commission will now be forthcoming from an authoritative source.

Times have changed and the cost of public works has notably increased. It is well, therefore, that there be this thorough investigation to settle the question. The country may look forward to receiving the report within a reasonable time, perhaps early in 1928. Then, there may be formulated a constructive program of the National Government as to the development of such a system of reservoirs and its effect locally and extending to the lower river. Perhaps it may be relied on as an example of what may reasonably

be done in other places, with similar conditions as to topography, construction possibilities, and relative effectiveness; to the end, in conjunction with other protective measures, that disastrous floods in the entire Mississippi Basin may be prevented forever.

CHARLES H. PAUL,* M. AM. SOC. C. E.—The work of the Miami Conservancy District has been referred to many times in the course of these discussions of the Mississippi River problems. It is significant, however, that it is only the retarding basins, or so-called reservoirs, of the District that have been mentioned. One fact seems to have been overlooked, that nearly one-half the construction cost of the Miami Flood Control System went into levees and river channel improvements. In other words, the Miami Conservancy project is a combination of retarding basins and of river channel improvements, the latter feature including the straightening, widening, and deepening of channels, and the construction or reinforcement of levee systems. The evolution of the plan was to accomplish as much by the channel improvement works as was economically feasible, and to rely on the retarding basins to take care of the surplus water that could not be handled within the limits of economy by further enlargement of channel capacities.

It has been stated by one or two previous speakers that the retarding basins of the Miami Conservancy District have had, or will have, little effect on the flood crests of the Mississippi, and there seems to be a tendency to use this as an argument against any use of reservoirs in connection with the Mississippi River control. It might easily be shown also that the levee system of the Miami Conservancy District will have no effect on the flow in the Mississippi, but certainly no one would go to the extent of giving this as an argument against the use of levees on the Mississippi, although that position would be about as reasonable as the other. The point is that the Miami Conservancy plan was designed solely for the control of the situation in the Miami Valley, and it would only be incidental if the work there had any effect one way or the other on the big problem of the Mississippi.

The Miami Conservancy project is fully effective for the object for which it has been designed (every one agrees to that), and this is so only because the plans were made after a thorough and comprehensive study of all necessary data. If there is one outstanding fact about the work of the Miami Conservancy District it is the thoroughness with which all necessary information was secured and studied before decision was made relative to the final details of the plan. This fact has impressed not only all former employees, but also all those who have studied the details of the plan. The speaker came into that situation after the general details of the plan had been decided upon; therefore, in commenting on the thoroughness of the preparatory work, he is speaking of the work of other engineers and not of his own. About two years were consumed by these preliminary studies, and all the necessary information was secured even though some of it was obtained with great difficulty, and at considerable expenditure of time and money.

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The outstanding necessity in connection with the solution of the Mississippi River problem is to secure all necessary information that has a bearing on the proper solution of this tremendous problem. Strange as it may seem, the basic information, such as is required for the development of a comprehensive plan, does not seem to be available. There is no possibility that it can be secured in a few months. It will take several years at best to secure the necessary data and to make intelligent analyses. It would be most unfortunate if the country were to become committed to a plan, and the large expenditures that necessarily would go with it, until after all such information is assembled and digested, and the best possible plan is developed. There is need, of course, for immediate appropriations to take care of the reconstruction of works that existed before the recent flood. Even such appropriations will run into large sums; but for the country to become committed at this time to a plan pretending to cover the whole situation, would be not only to risk the adoption of a plan that may not be adequate, but would be almost certain to result in wasting large sums that might be saved by a more deliberate and scientific study.

F. G. JONAH,* M. AM. SOC. C. E.—The recent flood in the Mississippi Valley wrought great damage to the railways in that region, and they are vitally interested in any plan for flood control that will minimize, if not wholly prevent, the tremendous losses that they sustain from these ever-recurring overflows.

There are accounts and records of many floods in the Mississippi River, the earliest being that given by Garcilasso de Vega, who wrote a history of De Soto's explorations. This he based on interviews with men who had been with De Soto on his great expedition. The flood occurred in 1543, and reading the account of it is very much like reading the account of any of the recent floods. One observation might be made with respect to that flood, and also of the others up to and including that of 1844, namely, that the forest growth then covered the entire valley, and apparently had no influence in lessening the flood. This is a fact that should be borne in mind when the project of reforestation is discussed.

Each succeeding flood will be, generally speaking, more destructive than the previous ones for the simple reason that there is more valuable property that can be destroyed. In 1882, when the railway lines first experienced trouble, there were about 200 miles of line in the delta. At present, there are 7 500 miles, and other developments have multiplied in proportion, towns, cities, improved highways, increased acres of land in cultivation, and a largely increased population.

From the earliest day of railway construction in the delta to the present there has scarcely been a year when the lines have not been subject to trouble from high water, and it appears that major floods may be expected to occur about once in every five years. The floods affecting railways seriously were those in 1882, 1890, 1892, 1897, 1903, 1912, 1913, 1916, 1920, and 1922. Certain lines sustained great damage, but in the flood of 1927, every road running

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through the valley below St. Louis, Mo., sustained enormous damages and losses. Due to the flood, train service was suspended on lines aggregating 3 000 miles, and in periods of from 10 to 120 days. The physical damage to these lines has been not less than $10 000 000, and in addition there is a loss, hard to estimate, growing out of the interruption of traffic and the general paralysis of business in the flood sections. These sections equal an area of 18 000 sq. miles, covering more than 100 counties and parishes, in several States, which is equivalent to the area of several States.

Unlike other calamities of Nature, such as earthquakes, cyclones, tornados, and tidal waves (which come without warning), a flood in the Mississippi Valley can be predicted, and with a remarkable degree of accuracy. The U. S. Weather Bureau and the Mississippi River Commission have predicted the passage of the crest of the flood and its highest stages at the several important points along the river to within a few hours of the actual time and a few tenths of a fôot of the actual heights attained. It is true that these predictions are revised from time to time during the flood, by reason of the breaks in the levee lines or continued and heavy rainfall. Thus, during the recent flood, there was an unprecedented rainfall over the water-shed of the Arkansas River, which brought that river to one of the greatest stages of record. This made necessary the revision of the predictions for the heights attained by the river at lower points, but these forecasts are so reliable that the railways know what to expect and make preparations accordingly.

The work of fighting the water during inundation is done in three general phases:

First.—To take all possible precautions to prevent overflow and to minimize the damage to track and structures when such overflow occurs.

Second.—To remove the inhabitants and as much of their household goods and live stock as possible from the overflow areas to places of safety.

Third.—To restore track and bridges to service and to resume train schedules in the shortest possible time.

The precautionary measures are the patroling and guarding of the levee lines, and building up the weakest spots with sandbags. In many cases the railways recruit large gangs of men for this work. They place them under the directions of the Government engineers and the municipal authorities, and they transport thousands of sandbags, planking, and laborers to those places that require this emergency work.

There is going on, at this same time, other precautionary measures, such as the cutting in of steam shovels for producing rip-rap, which is always needed in great quantities, and the assembly of work trains, extra gangs, pile-drivers, piling, ties, bridge timber, and rail and fastenings. The track is anchored where it is likely to wash off the embankments, and the ends of openings are protected with rip-rap.

The removal of the inhabitants and live stock is undertaken as soon as the danger is imminent. Many of the people remain until their lives are actually in danger. The railways haul them out to places of safety, furnish hundreds of camp cars and box cars for their temporary shelter, haul water and food-

stuffs for them, and provide, in many cases, doctors and medicine. After the flood subsides they haul them back to their homes, and haul seed for their planting, and these services are performed without charge. The railways co-operate with the Red Cross and municipal authorities very closely.

After these precautionary measures are taken, and, when the flood is on the lines, the first effort is to maintain communication as long as possible. Train service is carried on many days when the trains travel through water for many miles at slow speed, and then, after they can no longer run, the plan is to get the work trains in just as soon as the water goes down and restore the tracks for safe operation as quickly as possible.

The railway lines operating in that territory have developed a body of high-water fighters, who, by long experience, are capable of doing great work in these emergencies. Many miles of track are kept in service by raising it on slag, rock, or cribbing, securely anchored; but it is impossible for the companies to raise all their track above flood heights. The cost would be prohibitive. The delta country comprises about 30 000 sq. miles, in which there is now 7 500 miles of railway line, and the railway officials have long since concluded that it would be impossible to raise these lines above flood levels, and, were it possible to do so, they would not serve the country as well as they do now on their present elevations, because they are now adjusted to the industrial situation. Therefore, the railway companies rely on the protection of the levees and take their chances along with the other property owners in the affected districts.

It is not the speaker's purpose to discuss the measures that will be taken for the control of floods, as that subject is now being studied by the U. S. Corps of Engineers and the Mississippi River Commission. Their forthcoming report may be awaited with full confidence that it will contain many recommendations to Congress for practical flood control.

Since the last flood, there has been a flood of literature and discussion in the technical journals and the public press, advocating every conceivable method of flood control. There have been some suggestion of impounding reservoirs or detention basins in the valley itself. It has been suggested that the St. Francis Basin be used for this purpose, and this entire area of 4 000 sq. miles converted into a catchment basin. Some mention was made of the possibility of this as far back as the report of Humphreys and Abbot, in 1861, but at that time it was swamp land covered with primeval forest growth. It was public domain and the lands could have been segregated by the Government, but now conditions are different. This land comprises the most fertile region of Southeast Missouri and Northeastern Arkansas, in fact, the best agricultural portions of those two States. Statistics will show that while certain counties in these States either lost population, or were stationary in growth, in the last two decades, the counties in the St. Francis Basin rapidly increased in population. It is now covered with prosperous cities and towns, farms on every 160 acres almost, and it is one of the few spots in the world where wheat, corn, cotton, and rice can be grown on adjacent fields. Within this basin there are 800 miles of railway lines, and hundreds of miles of

concrete highways. These lines could not be abandoned under any circumstances, as they are necessary to keep up communication between Memphis, Tenn., and the great region to the west. The cost of raising any of these lines above the suggested reservoir level would be absolutely prohibitive.

It seems a more reasonable solution for relief in the St. Francis Basin would be to make a controlled spillway below Cape Girardeau, Mo., leading the water into the St. Francis River, putting levees on this river as well as on the Mississippi. This water, of course, would again reach the Mississippi at Helena, Ark., but it could be carried around in this new channel or by-pass much more quickly than it would come down the main channel of the river, a much longer distance, and thus the crest of the flood might be divided or broken.

A part of the flood waters of the Arkansas might be diverted through a controlled spillway to the Tensas Basin and down the Tensas and Black Rivers to the Red River and the Atchafalaya, through which the Red River discharges when the Mississippi is at flood stage. The main channel might thus be greatly relieved by the diversion of these flood waters of certain of its tributaries.

The settlement of the Mississippi River Valley proceeded along the river itself. The towns and villages are built close to the river bank, because when the country was settled the only means of communcation was by the river. Traffic was carried by the steamboats, and towns grew up around the steamboat landings. The highly developed farms front on the river bank because the lands are higher there than elsewhere in the delta. The highways are along the river bank, and the railway lines are likewise very close to the river as they run from town to town. In the proposal of setting certain portions of the levee lines back from the edge of the bank, the railway lines will have to be adjusted to new conditions. There is no doubt that many miles of levee have been built too close to the bank and are subject to the scouring and undermining of the river currents.

It is assumed, of course, that New Orleans must be protected at all hazards, and it would seem that the city can be made secure by converting the Atchafalaya into a permanent controlled spillway, opening up another one through Bayou Manchac, Lake Marepas, and Lake Ponchartrain, and the removal of many miles of levee lines below the city. As emphasized in Colonel Townsend's admirable paper,* the present law creating the Spillway Board, prohibits the discharge of overflow water into Mississippi Sound. This doubtless is due to the objection that New Orleans has always had to the discharge of silt into Lake Ponchartrain, but many engineers believe that it would be a very long time indeed before this occasional discharge would effect the lake. The silt would be deposited in Lake Maurepas and in the surrounding swamps, and by the time the discharge reached Lake Ponchartrain it would be comparatively clear. This Act could be amended when conditions warrant it.

* *Proceedings*, Am. Soc. C. E., December, 1927, Papers and Discussions, p. 2454.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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PROBABILITY OF FLOOD FLOWS

Discussion*

By Ralph R. Randell, M. Am. Soc. C. E.

Ralph R. Randell,† M. Am. Soc. C. E. (by letter).‡—The methods developed in the paper§ by H. Alden Foster, Assoc. M. Am. Soc. C. E., and applied by the author to determinations of the probability of flood flows, consist essentially of two operations: First, the development of a theoretical equation which, for the data of record, more or less approximately represents the relationship between the magnitudes under investigation and the frequency with which they occur; and, second, the application of that equation to periods and values beyond the limits of the observations. In general, such methods are liable to error in that (1) the theoretical equation may not express with sufficient accuracy the true relationship during the period and within the range of the observations, as determined by the data of record; and (2) the relationship for other periods or ranges may not follow the same equation or laws.

The particular types of theoretical equations which are utilized in the methods under discussion were derived from a general formula by a process involving certain assumptions which permit the development of workable equations but limit their applicability. The mere fact that a curve representing one of these theoretical equations approximately fits a given set of observations is no assurance that it also will represent correctly extensions of the corresponding true curve. Various types of curves are approximately similar over parts of their lengths but quite dissimilar over other parts. Portions of a circle, a parabola, or an ellipse might represent a given limited set of plotted points more or less equally well; yet if observations covering a wider range were available these curves might be found to represent the extension with widely varying degrees of fidelity, the one best fitting the original points per-

* Discussion of the paper by F. G. Switzer, Assoc. M. Am. Soc. C. E., continued from October, 1927, *Proceedings*.

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‡ Received by the Secretary, November 12, 1927.

§ *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), p. 142 *et seq.*

haps departing farthest from the true extension. These uncertainties of the methods used are well illustrated by the questions and doubts that have arisen as to whether Type I or Type III curves* are most appropriate for use in analyses of given sets of data.

Precipitation and stream flow data have definite lower limits, and the magnitudes which occur most frequently are less than the means. The methods under discussion present only two general types of frequency curves, I and III, appropriate for the study of phenomena having these characteristics. Each of these types is represented by a relatively simple formula, which in each case has but three constants subject to selection to make the formula fit the particular phenomena under investigation. In the practical application of either of these two formulas, or types of curve, to the determination of theoretical duration curves for phenomena of the character mentioned (which is their principal field of possible usefulness to the hydraulic engineer), two factors, *cv* and *cs*, alone fully fix and determine the shape of the theoretical duration curve applicable to any given set of data. Of these only one, *cv*, generally needs to be fixed with any degree of accuracy. The practical utility of these methods for investigations of such phenomena is limited, therefore, to cases in which one of two relatively simple type-curves may be made to fit with sufficient exactness both the observed data and the most probable extensions thereof, solely by the appropriate selection of only one accurate and one approximate constant.

It is improbable that precipitation or stream flow data can follow such relatively simple formulas or curves with even approximate correctness over any considerable range. This is apparent from a consideration of the variability and other characteristics of some of the important factors affecting the amounts and distribution of precipitation and stream flow. Among such factors may be mentioned the various planetary cycles, all of which are variable with respect to time and different in periodicity and magnitude of influence. Together with their phase inter-relationships, these cycles cause important climatic changes which, for any period likely to be used in practice, vary erratically and not according to the laws of probability. Unless appropriate adjustments are made, such changes, occurring during any period of record, impair its homogeneity. Where they extend beyond the period of record into the future to be estimated, they vitiate any estimate based on relationships existing during such period. In view of the erratic and relatively great influence of these planetary cycles it is not to be expected that mere multiplicity of influencing factors will introduce others which will synchronize with them so as to counterbalance (even approximately) their essential unsusceptibility to practical analysis by probability methods.

The influence of storage on stream flow is another example of important factors which may not adequately be represented by such simple formulas, having such limited means of adjustment to fit various circumstances. Lakes in a tributary water-shed, for example, may appreciably affect one part of a duration curve and not another. Various other influencing factors, such as

* *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), pp. 148 and 182; also, *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, p. 565.

the likelihood of coincidence of humidity and temperature conditions favoring rain, affect the whole curve. Obviously, no single constant can represent with fidelity two such different types of influences, one of which affects the whole curve and the other only a part. As would be expected from the foregoing, none of the theoretical curves shown as examples by the author or by Mr. Foster* fits the observations as well as would some other curve that could be drawn by eye. Indeed, with phenomena that vary so complexly and from such variable causes, it is surprising that, solely by the appropriate selection of only one or two constants, such relatively simple theoretical curves can be made to fit as many sets of observations as they do, as well as they do.

Curves drawn by eye to follow closely the actual observations generally reflect the complexity and variability of the influencing factors by their relative irregularity as compared with the theoretical curves. It is true, of course, that, unlike the theoretical curves, such relatively irregular curves do not present any simple, obvious, and constant relationship by which they may be extended with mathematical definiteness beyond the limits of observation in the most probably correct manner. This is as it should be, however, inasmuch as with only the present limited knowledge the most probable magnitudes for periods beyond the range of observations actually are indeterminate for the reasons mentioned elsewhere herein, and fidelity requires that any curve purporting to represent the true conditions shall reflect this. The apparent definiteness of the theoretical curves is, for the same reasons, merely an illusion. The departures of the theoretical curves from the plotted observations are generally most marked at and near the limits of the region covered by the data. The agreement in these vicinities especially is not such as to inspire confidence in the theoretical curves even within the range of observations, let alone beyond this range, and especially far beyond.

Even aside from the effects of cycles and the uncertainties of estimating curve extensions on determinations of future or long-term probabilities, examples are not lacking of the danger of applying observed laws beyond the limits of the observations from which they have been deduced. Witness, for example, the familiar vagaries of the variation in the volume of water with changes of temperature. Who, observing the relationship between these variables only between 10° and 90° cent., for example, would suspect that it would be reversed between 0° and 4° in a manner so apparently contrary to usual natural laws? Who is to say that no comparable incongruity may exist in precipitation or stream-flow frequency or duration curves beyond the field of observation in the realms of speculation, which unwisely may be probed by the methods under discussion?

Finally, aside from the weaknesses and unreliability of such methods, their practical application has its difficulties and limitations. As the author suggests,† they might be applied to various problems on a basis of the floods that would probably occur during a period equal to the expected life of the structures involved, or on a basis of the relative economies of building to withstand floods that were to be expected once during periods of various lengths. The

* *Transactions*, Am. Soc. C. E., Vol. LXXXVII (1924), pp. 168–170.

† *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, p. 567.

critical flood, however, might be exceeded to-morrow for all any one knows or can determine, so that such applications of these methods would at best be difficult, and would perhaps involve complicated considerations of insurance as to the particular time during any given period at which a maximum flood would actually occur. In any event, such applications could not well be made to problems involving the safety of human beings, such as the spillway capacity to be provided for dams above towns of settlements, for who would set a value on human life? No ultimate maximum for use in such cases can be obtained from curves of Type III, regardless of how long a period is selected; and, for the reasons set forth, maxima obtained from curves of Type I would be so uncertain as to be of doubtful value.

For the reasons stated, the methods under discussion (although apt, on account of their mathematical definiteness, to convey a false impression of truth and accuracy) are practically valueless for estimates involving very long periods. For the same reasons, it is felt that, for periods of moderate length, they are not likely to lead generally to as probably correct results as the much simpler methods of plotting and extending the original duration curves by eye and seasoned judgment. This is especially true where such plotting and extension is done on some form of paper, such as probability or logarithmic, that exaggerates the extremes to be extended and makes the curves flatter. The methods are undoubtedly elegant from a mathematical standpoint, and, if used with full appreciation of their limitations and shortcomings, may perhaps have some legitimate value as a guide to judgment. It is believed, however, that of themselves they are essentially liable to untruth and inaccuracy and are dangerously deceptive in their seeming correctness and exactness. All things considered, it is felt that future advances in the knowledge of long-term precipitation and stream-flow conditions must be along the lines of enlarging the understanding of the planetary and other fundamental causative influences bearing on climate, rather than in greater mathematical precision or apparent rationality in the analysis and extension of present or future records.

The evidence which the author presents,* that floods vary approximately as the 0.6 power of the drainage area, is very interesting, and constitutes a welcome addition to the rather brief literature on this subject. As the author suggests, these conclusions are justified, of course, only to the extent that the streams used in deriving and applying the relationship are similar with respect to water-shed characteristics and other flood-influencing factors, except only area of water-shed.

The author merits the sincere appreciation of engineers interested in problems of flood flows for his clear-cut exposition of applications of the theoretical frequency-curve methods of analysis.

* *Proceedings*, Am. Soc. C. E., April, 1927, Papers and Discussions, pp. 566–568.

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TIDES AND THEIR ENGINEERING ASPECTS

Discussion*

By Messrs. F. W. Scheidenhelm, and Charles H. Lee and Harold F. Gray.

F. W. Scheidenhelm,† M. Am. Soc. C. E.—The respect which the speaker has had for the work of the U. S. Coast and Geodetic Survey has not been diminished in the least by this paper. Indeed it has been broadened to include the well-nigh human tide-predicting machine devised and designed by the engineers of the Survey.

The speaker is especially interested in the engineering aspects of tides as they affect water-power development. Much has been said and written about the proposed development of tidal power in the Bay of Fundy, and in regard thereto the speaker joins in marveling at the courage, and likewise the optimism, of those who seek to curb and utilize those tides for the generation of hydro-electric power on a mammoth scale. He has in mind, not the generation of power by means of tides, but rather the effect of tides on the more usual type of water-power development along streams.

At St. John, the principal city and seaport of the Province of New Brunswick, Canada, located on the Bay of Fundy, the outstanding natural attraction is the so-called "Reversible Falls" at the mouth of the St. John River. That river drains about 26 000 sq. miles and hence under ordinary circumstances would afford an attractive opportunity for the development of water power at the Falls. However, the foot of the Falls is practically at low tide, and, despite the fact that at times a fall of perhaps 12 ft. is apparent, the tides of the Bay of Fundy of about 20 ft. completely submerge the Falls at high tide. Indeed, the flow is then actually up stream, sometimes to a depth of about 12 ft. over the crest of the falls. For a period of less than an hour, at high slack tide, ships can pass from St. John Harbor over the falls up into the river.

* Discussion on the paper by G. T. Rude, M. Am. Soc. C. E., continued from December, 1927, *Proceedings*.

† Cons. Engr. (Mead & Scheidenhelm), New York, N. Y.

This condition makes it apparent why the Falls by themselves are not suitable for the development of hydro-electric power. However, inasmuch as the development of tidal power in the ordinary sense involves shifting the means of development according to the ebb or flow of the tide, it may not involve too great a flight of the imagination to conceive that at some future time the Reversible Falls at St. John may be called upon to function in the generation of power for a part of each 24-hour period, with corresponding reliance placed upon some other source of power generation during periods of partial and complete submersion of the Falls.

In passing, it may be noted that the tidal fluctuations of the Bay of Fundy take place to the full extent in the Harbor of St. John and give rise to problems affecting harbor construction, the use of docks, and the handling of shipping. These problems have many interesting and difficult phases. The situation may be the better understood if one will visualize the fact that at high tide a vessel alongside a dock rides about 20 ft. higher than at low tide.

The speaker has had occasion to investigate the Provincial development (by the New Brunswick Electric Power Commission) of the power of the Musquash River, which empties into the Bay of Fundy about 15 miles west of the City of St. John. Inasmuch as the power station is located close to sea level, the great tides of the Bay play an important part. They affect the power-station tail-water for about 15 hours of each day. The power project really consists of twin developments, in each of which an intake reservoir is connected by a wood-stave pressure pipe to the power station. The twin developments operate under average effective heads of about 100 ft. and 120 ft., respectively, and the sources of water supply are not interchangeable.

In the usual type of water-power development the head is at a minimum during flood periods. In the case of the Musquash, however, the heads, so far as tail-water is concerned, are at a minimum at high tide. Naturally, it is the spring tides, as described by the author,* that are the most serious in this respect. At times, the spring tide at St. John attains at least 28 ft., or about 8 ft. in excess of the relatively high normal tides.

When a spring tide coincides with a depleted condition of an intake reservoir, it follows that there will be reduction of head both at the head-water and at the tail-water. In order to maintain the turbine output at approximately normal capacity, it would be necessary to utilize greater water supply with correspondingly greater velocity of flow through the pipe line. This, in turn, causes greater friction loss and, consequently, further loss in energy.

So far as the reduction due to tides is concerned, conditions may be further aggravated if the wind should happen to be in the proper direction to heap up the tide still more. Indeed, it is not impossible that the combination of the several tidal components may result in increasing all tides, spring as well as normal, in their travel up the Musquash Estuary at the head of which the power station is located. As concerns the Musquash water-power development, it seems fortunate that it is the east and not the west side of the Bay of Fundy that has the 5-ft. higher tides. Another interesting phase of the

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1079.

Musquash situation is that, during the period of tidal records to which the speaker had access, there appeared to be no instance of an extreme spring tide occurring between 4:00 and 6:00 P. M. These are the hours of coincidence of industrial demand for power with the needs of power for early evening lighting during the autumn and winter, and thus they constitute the period of maximum demand on the power station. It may be, therefore, that, during that critical period of the day, the power station may be spared the reduction of its head due to the full extent of the maximum spring tides. As to whether or why maximum spring tides never occur during that period of the day, the speaker has no knowledge, nor did his inquiries meet with any satisfactory answer.

CHARLES H. LEE,* AND HAROLD F. GRAY,† MEMBERS, AM. SOC. C. E. (by letter).‡—The writers have been especially interested in the flow of tidal currents in estuaries and bays. In connection with the question of sewage pollution they have had to determine the rates of discharge and volumetric renewal of water in the tidal canal known as the Oakland Estuary lying between the Cities of Oakland and Alameda, Calif.

Description of Conditions.—Referring to Fig. 45, the estuary is seen to extend as a dredged channel from San Francisco Bay at its northwestern end, between the Southern Pacific (Alameda) and the Western Pacific Railway moles, to San Leandro Bay at its southeastern end. San Leandro Bay is a shallow body of salt water, much of its area being mud flats at low tide. It has a southerly opening into San Francisco Bay between Alameda and Bay Farm Island.

The level of water in the Estuary is promptly affected by tide-level changes in San Francisco Bay, and strong currents occur during each ebb and flow period. The filling and emptying of the tidal prism in San Leandro Bay largely results from tidal flow through the estuary. The tidal currents at the San Francisco Bay outlet of San Leandro Bay (at Bay Farm Island Bridge) are seldom strong, and are inward more than 75% of the time. Outward flow occurs at irregular intervals and during four to six hours each day, being controlled by the relative levels of San Francisco and San Leandro Bays at the Bay Farm Island Bridge. These relative levels depend, in turn, on the filling or emptying of San Leandro Bay through the tidal canal of the Oakland Estuary.

At present, about one-half the sewage of the City of Oakland, and most of the sewage of the City of Alameda, discharges into the Oakland Estuary. The problem was to determine the probable future date at which the quantity of sewage discharged therein would be so great as to exceed the safe dilution capacity of the Estuary water.

The total volume of water in the Estuary for various tide stages between the end of the Alameda Pier and the High Street Bridge, and between the High Street and Webster Street Bridges, was determined by computations from cross-sections furnished by the Corps of Engineers, U. S. Army (First

* Cons. Hydr. Engr., San Francisco, Calif.
† San. and Hydr. Engr., Berkeley, Calif.
‡ Received by the Secretary, November 11, 1927.

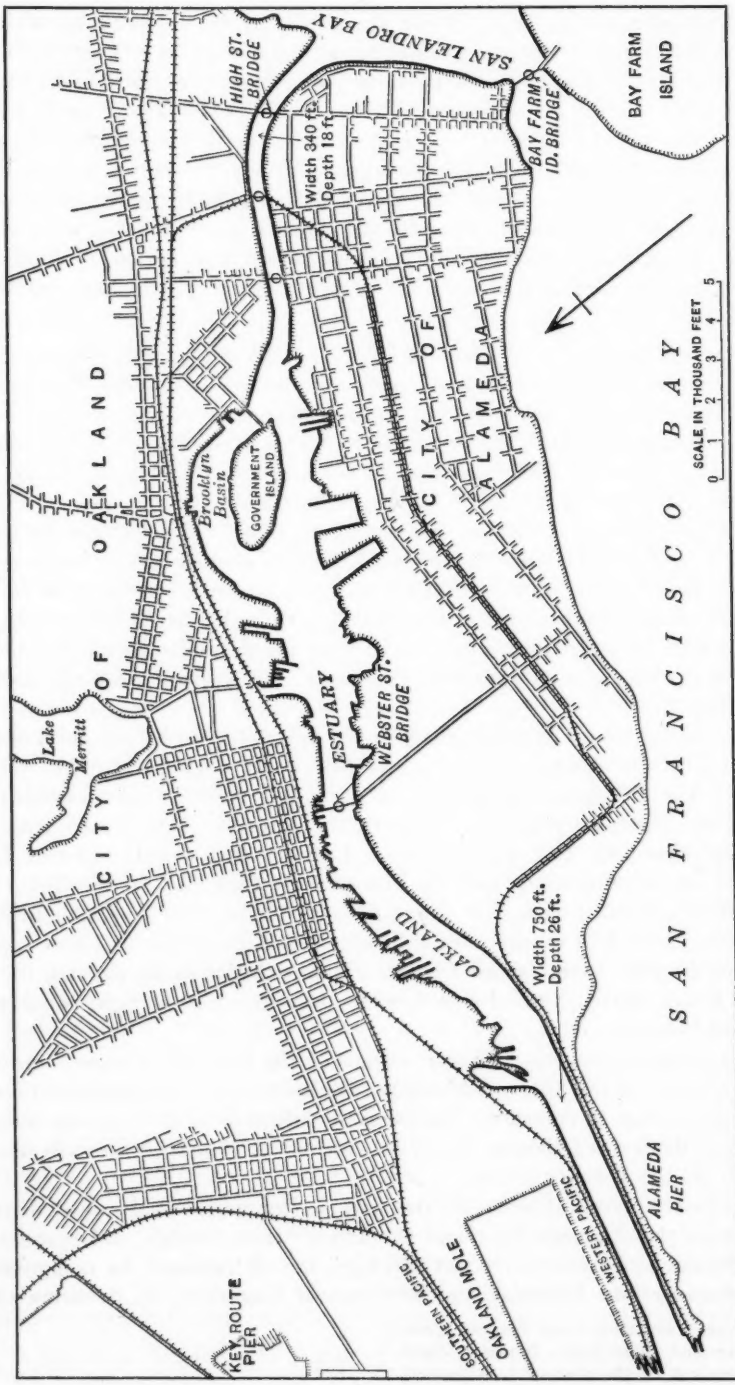

FIG. 45.—OAKLAND ESTUARY, OAKLAND, CALIF.

San Francisco District). Curves showing such tide stage-volume relationships are presented in Fig. 46. Tidal discharges were obtained by measurements with current meters at the High Street Bridge for various ranges of tide. Check measurements were made during the first set of observations by simultaneous current-meter gaugings at the Webster Street Bridge. Velocity measurements by means of rod floats (weighted lath) were also made at Bay Farm Island Bridge (where the velocities were too low to be measured by current meters), and tide stages and current directions were observed at all four stations.

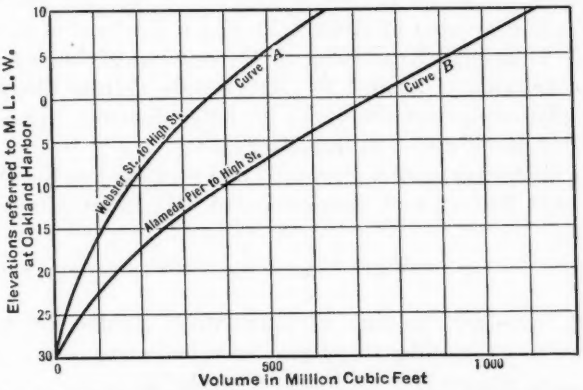

FIG. 46.—CAPACITY-TIDE ELEVATION CURVES, OAKLAND ESTUARY.

Program of Observations.—Careful soundings were first made at the High Street, Webster Street, and Bay Farm Island Bridges to determine the cross-sections and to designate the current-meter stations for both incoming and outgoing tides. Staff gauges were set at these bridges and illuminated so as to be easily read at night. An automatic water-stage recorder was set at the end of the Alameda Pier.

Preliminary to the actual gaugings, the tide gauges were observed for several days to determine whether there were any marked peculiarities of flow to be considered. Current directions under the bridges were observed by suspending the meter in the water and measuring the angle between the bridge rail and the tail of the meter. At the High Street Bridge the direction of flow was at a marked angle, and this had to be taken into account in the computations.

The results of the first set of measurements indicated that discharge measurements at the High Street Bridge would give all necessary information, and check measurements at the other bridges, therefore, were not made at the times of subsequent observations.

The first set of discharge measurements was made on August 12 and 13, 1927, when a large tidal range was anticipated. The following schedule was observed:

1.—An observer was stationed at Alameda Pier to note the times of change of current direction.
2.—An hydrographer and assistant were stationed at the Webster Street Bridge, with a current meter.

3.—An hydrographer and assistant were stationed at the High Street Bridge, with a current meter.

4.—An observer with two assistants was stationed at the Bay Farm Island Bridge, to measure velocities by means of rod floats (weighted lath).

Continuous tide-gauge observations were made, and discharge measurements were repeated as frequently as possible during the tidal runs. There were numerous interruptions due to openings of the draw-spans at the Webster Street and High Street Bridges. The observations covered a tidal range of 5.0 ft. on the incoming tide from high low water to high high water on the afternoon and evening of August 12, and a tidal range of 8.4 ft. on the outgoing tide from high high water on the evening of August 12 to low low water on the morning of August 13. The results of these discharge measurements were plotted against time, and the total discharge for each tidal run was determined graphically (see Fig. 47(*a*)).

The total discharges as thus measured and computed at the Webster Street and High Street Bridges were then compared with the results computed from the formula,

$$Q_1 = (V_1 - V_2) + Q_2 \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

in which,

Q_1 = volume passing Webster Street Bridge, or Alameda Pier, according to purpose of calculation;

Q_2 = volume passing High Street Bridge;

V_1 = volume in Estuary between bridges at higher tide stage; and

V_2 = volume in Estuary between bridges at lower tide stage.

For the incoming tide on August 12,

Q_2 = 121 000 000 cu. ft. (measured)

$V_1 - V_2$ = 106 565 000 " "

Q_1 = 227 565 000 " " (computed)

Q_1 = 228 000 000 " " (measured)

For the outgoing tide on August 12-13,

Q_2 = 197 800 000 cu. ft. (measured)

$V_1 - V_2$ = 173 152 000 " "

Q_1 = 370 952 000 " " (computed)

Q_1 = 368 000 000 " " (measured)

As the agreement between the measurements and the computed values at the Webster Street and High Street Bridges were thus reasonably close, subsequent measurements of flow were made at the High Street Bridge only.

The discharge curves for subsequent observations made on September 6 and October 4, 1927, are shown in Fig. 47 (*b*). The tide range on August 12 and 13 was large; that on September 6 was intermediate; and that on October 4 was small.

The total discharges for these tide runs of varying range as obtained from Fig. 47 are shown on Fig. 48. The plotted points indicate a direct relation between tidal range and discharge, as shown by the straight line. At low

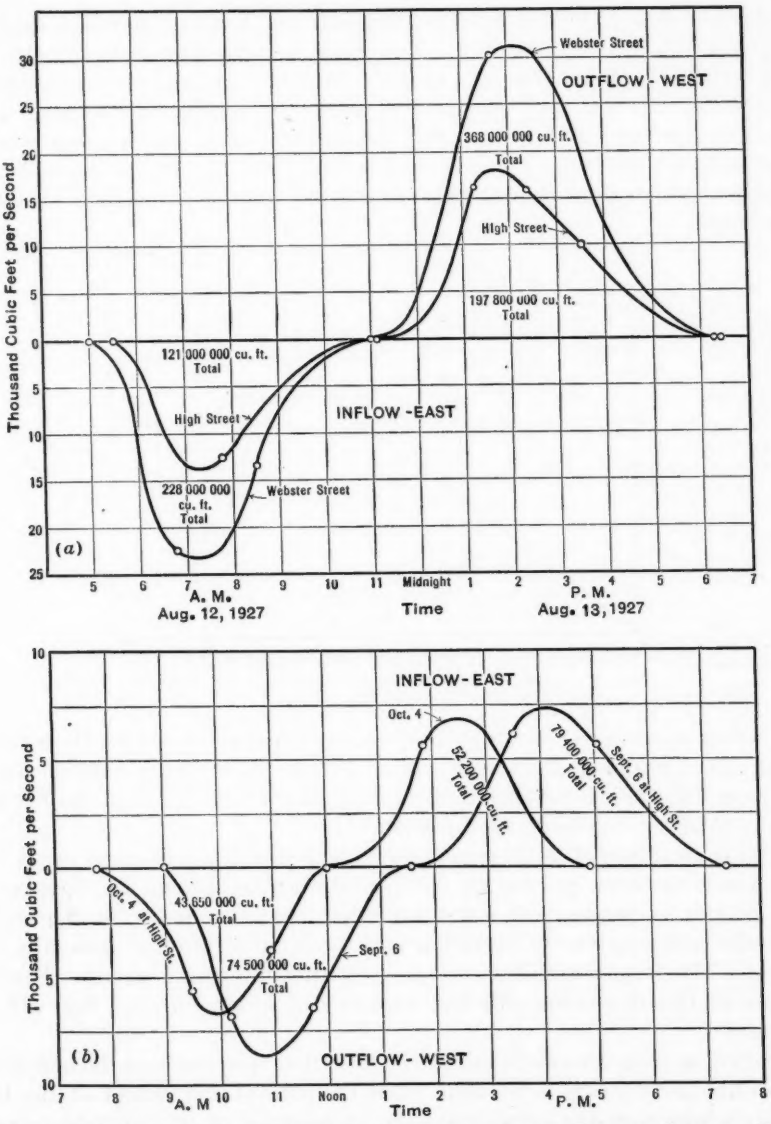

FIG. 47.—TIDAL FLOWS AT HIGH STREET AND WEBSTER STREET BRIDGES, OAKLAND, CALIF.

ranges the tidal flow west at High Street Bridge for the same tidal range is somewhat erratic. This is probably due to hydraulic conditions in San Leandro Bay, which has a flow out into San Francisco Bay only at short and irregular intervals. It is probable that this condition assumes importance only at small tide ranges. In view of its irregular occurrence it has been neglected in this investigation.

The inflow through the Alameda Pier entrance was the only one considered to be of immediate concern, for the reason that dissolved oxygen tests

on the ebb flow from San Leandro Bay into the Estuary showed a depletion of about 50% below saturation. The water coming from San Leandro Bay and flowing down the Estuary on the ebb tide is, therefore, so depleted in oxygen that for practical purposes it cannot be considered as "new" water available for sewage dilution. For the calculation of replacement percentages, only the incoming water from San Francisco Bay at the Alameda Pier entrance, which is constantly at or close to oxygen saturation, should be considered.

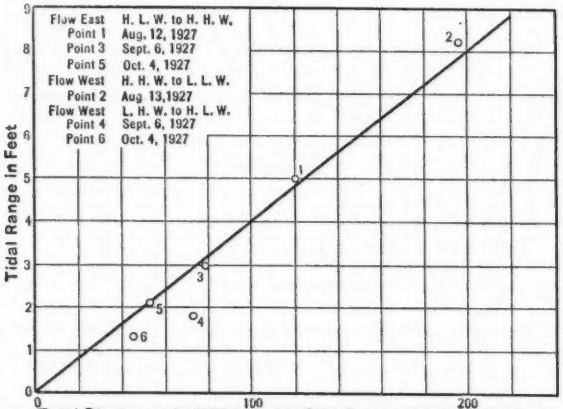

FIG. 48.—TIDE RANGE-DISCHARGE CURVE, HIGH STREET BRIDGE, OAKLAND, CALIF.

Having determined the tide stage-discharge relationship for the High Street Bridge, as shown on Fig. 48, and having the tide stage-volume relationship as shown on Fig. 46, it is possible to determine the total tidal flow in the Estuary for any tidal run by means of Equation (1).

The critical period with respect to sewage dilution will occur when the tidal flow is the least, and during that period when the sewage has the greatest total oxygen demand. The latter condition occurs annually in September when the canneries are in operation and the temperatures are generally the highest. The former condition occurs approximately once a month. For the purpose of this discussion only the condition of minimum tidal flow will be considered.

Excluding from consideration the ebb flow, it is apparent that the minimum inflow into the Estuary will occur when the sum of the ranges of the two incoming tides each day is the least. An examination of the tide tables shows such a minimum range on September 2 and 3, 1927, as an example. These ranges at the Fort Point standard station for San Francisco Bay are as follows:

High low water to high high water..	2.9 ft. to 4.9 ft.	=	2.0 ft.
Low low water to low high water...	1.5 ft. to 3.8 ft.	=	2.3 ft.
Total.....		=	4.3 ft.

The U. S. Coast and Geodetic Survey's factor for tide ranges at Oakland Harbor (Session's Basin) is 1.22, to be applied to the ranges at the Fort

Point standard station. Applying this factor to the ranges given, there results 2.4 ft. and 2.8 ft., respectively, or a total of 5.2 ft. Using these values and taking off corresponding discharge and volume quantities from Figs. 46 and 48, for high low water to high high water, 2.4-ft. range,

$$\text{At } (4.9 \times 1.22) = 6.0\text{-ft. stage, } V_1 = 955\,000\,000 \text{ cu. ft.}$$
$$\text{At } (2.9 \times 1.22) = 3.5\text{-ft. stage, } V_2 = 865\,000\,000 \text{ cu. ft.}$$
$$Q_2 = 60\,000\,000 \text{ cu. ft.}$$

Therefore,

$$Q_1 = (955\,000\,000 - 865\,000\,000) + 60\,000\,000 = 150\,000\,000 \text{ cu. ft.}$$

For low low water to low high water, 2.8 ft. range,

$$\text{At } (3.8 \times 1.22) = 4.6\text{-ft. stage, } V_1 = 905\,000\,000 \text{ cu. ft.}$$
$$\text{At } (1.5 \times 1.22) = 1.8\text{-ft. stage, } V_2 = 805\,000\,000 \text{ cu. ft.}$$
$$Q_2 = 70\,000\,000 \text{ cu. ft.}$$

Therefore,

$$Q_1 = (905\,000\,000 - 805\,000\,000) + 70\,000\,000 = 170\,000\,000 \text{ cu. ft.}$$

The total inflow at Alameda Pier will then be the sum of the two values of Q_1, or,

$$150\,000\,000 + 170\,000\,000 = 320\,000\,000 \text{ cu. ft.}$$

The volume of water in the Estuary between Alameda Pier and High Street Bridge at mean sea level is 863 000 000 cu. ft. Therefore, the approximate percentage of new water entering at Alameda Pier is,

$$\frac{320\,000\,000}{863\,000\,000} = 37$$

If, instead of calculating the percentage on the basis of the average tide stage (mean sea level), it is calculated on the basis of the volume of water in the Estuary at the beginning of each inflow, the percentages would have been 17 and 21, respectively, or a total of 38 of new water. On this basis one would expect a complete change of water in the Estuary every 2½ to 3 days (except at dead ends and in pockets alongshore) under the worst probable conditions.

As all field work was done according to standard practice and under reasonably good working conditions, the actual measurements of discharges and volumes are believed to be within reasonable limits of accuracy (possibly $\pm$ 5%). In drawing the graphs of the tidal flows (see Fig. 47), it was necessary to exercise more or less judgment in order to draw curves which would represent the actual flows satisfactorily. However, a reasonable estimate of the volume of tidal flow available for sewage dilution has been obtained. This estimate, combined with information as to the present and probable future total oxygen demand of the sewage coming into the Estuary and San Leandro Bay, the amount of surface re-aeration, and other significant factors, should serve as a fair basis for the estimation of future conditions in this Estuary.

This investigation was a part of the studies being made by the Engineering Commission on Sanitation and Drainage of the City of Oakland, of which Charles D. Marx, Hon. M. Am. Soc. C. E., and the writers are members.

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PAPERS AND DISCUSSIONS

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NOTES ON ARCHED GRAVITY DAMS

Discussion*

By Messrs. Paul Bauman, William Cain, F. W. Hanna, E. W. Kramer, A. J. Wiley, W. P. Creager, Edward Godfrey, and C. A. P. Turner.

Paul Bauman,† Assoc. M. Am. Soc. C. E. (by letter).‡—Valuable equations have been derived by the author, which in a large measure supply the need for simple and yet accurate formulas for the determination of cantilever stresses in arch dams. Certain general conclusions have been advanced, however, which involve more or less error. A measure of the advantage or disadvantage of building a gravity dam, curved in plan, can only be determined after careful study of the topographic conditions.

A gravity dam is necessarily a very massive structure, and, due to the manner in which it is invariably built, the analysis of a thin vertical slice is insufficient. Consideration must be given to the effect of arch action and even of wedge action where the dam is built in narrow canyons. In principle, one might say that the wider the valley the less advantage there would be in a curved dam, assuming always equal foundation conditions along either a straight or curved location.

The possibilities of design of any massive dam lie between the limiting conditions of a straight cantilever dam of infinite length and a true arch dam. Under these ideal, limiting conditions there is either cantilever action or arch action only. In all practical cases, however, there will always be secondary cantilever action or secondary arch action, the amount in each case depending mainly on the ratio between the height and length, and the thickness and curvature, respectively.

It is the writer's opinion that there is insufficient justification for ignoring the effect of arch action in the case of a curved gravity dam, such as the

* Discussion on the paper by B. F. Jakobsen, M. Am. Soc. C. E., continued from November, 1927, *Proceedings*.

† Designing Engr., Quinton, Code & Hill, Los Angeles, Calif.

‡ Received by the Secretary, October 24, 1927.

Exchequer Dam. Without question the actual arch action will not be in strict accordance with the computed arch action, but much less error will be involved in considering arch action than in eliminating it from consideration.

The author probably makes the assumption that, because of shrinkage in the concrete, contraction joints open and thus prevent any arch action. Compensating this shrinkage are other factors, such as plasticity and the swelling of the concrete due to absorption of moisture. Concrete has been known to flow even several years after the dam has been completed. Furthermore, contraction joints are not mathematic surfaces, and, as long as sufficient area is in contact to permit transmission of arch thrust without crushing, such arch action will be developed.

The writer has made an analysis of the stresses which, in his opinion, must closely approximate the actual stresses in the Exchequer Dam. The results of this analysis show the effect of arch support, and the conclusion is that no tensile stresses exist at the up-stream face, as claimed by the author, and that appreciable compressive stresses do occur instead, especially in the lower portions of the dam.

The radial cross-section of the dam was taken from *Western Construction News* of January 25, 1926, and the length of the up-stream face from Sheet No. 9 of the Merced Irrigation District, dated June, 1924. The water surface was assumed to be at the top of the dam at Elevation 710.0, and the up-stream radius, or R_u, constant at 675.0 ft. The notation used in the following analysis is the same as that used by the author with the following additions:

t = radial thickness of dam.
h = head of water above horizontal sections.
L_u = length of extrados.
S = horizontal shear.
S'_A = deflection of arch slice $= C \frac{p\, r^2}{E\, t}$ (due to full load).*
δ_A = deflection of arch slice due to partial load.
$\left.\begin{matrix}\delta'_B \\ \delta'_S\end{matrix}\right\}$ = deflection of cantilever due to bending and shear, respectively, due to full load.
$\left.\begin{matrix}\delta_B \\ \delta_S\end{matrix}\right\}$ = deflection of cantilever due to bending and shear, respectively, due to partial load.
S_u = section modulus of up-stream face.
S_d = section modulus of down-stream face.
K_u = distance from center of gravity of horizontal section to down-stream limit of "kern".†
K_d = distance from center of gravity of horizontal section to up-stream limit of "kern".†
H_0 = polar distance of force polygons.
E = modulus of elasticity of concrete = 2 500 000 lb. per sq. in.

* "The Circular Arch Under Normal Loads," by William Cain, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXV (1922), p. 233.

† The "kern" of a section lies within the line as described by the point of application the neutral axis of which is a tangent on the section.

For the determination of δ'_B and δ_B, Mohr's method was used,* whereas for the determination of δ'_S, the least work method was used, as follows:

$$\delta'_S = \int_0^H \frac{S' S}{G A} d x$$

in which, $G = \frac{5}{13} E$ and $S' = -P = -1$ (Fig. 8). Therefore,

$$\delta'_S = -\frac{1}{G} \int_0^H \frac{S d x}{A} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (28)$$

$$S = \frac{62.5 (H - x)^2}{2}; A = \frac{B}{H} (H - x)$$

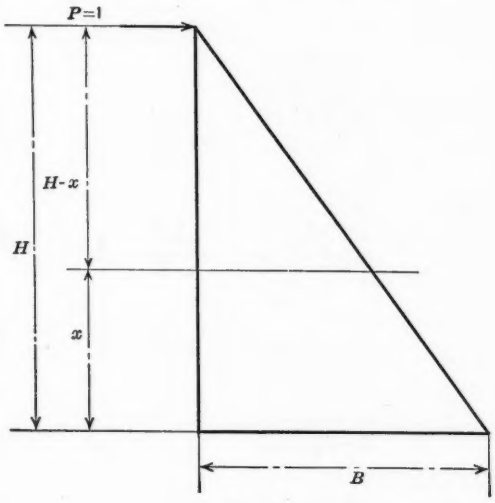

FIG. 8.

Introducing these values in Equation (28),

$$\delta'_S = -\frac{31.25 H}{G B} \int_0^H (H - x) d x = -\frac{31.25 H}{G B} \left[H x - \frac{x^2}{2} \right]_0^H + c (= 0) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (29)$$

For the determination of δ_S (partial load) the $\frac{S}{A}$-area, Fig. 9 (*e*) was graphically integrated and multiplied by $\frac{I}{G}$; that is, Equation (28) was used in its form:

$$\delta_S = -\frac{I}{G} \sum_0^H \frac{S}{A}$$

In Table 1 various quantities, determined by the author's formulas, are shown. The vertical weights, W (based on a triangular section), their lever arms, $(y_a - Y_w)$, the distance, Y_a, from the up-stream face to the center of

* "Gravity and Arch Action in Curved Dams," by F. A. Noetzli, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 1.

TABLE 1.—ANALYSIS OF EXCHEQUER DAM.

Elevation, in feet.	h, in feet.	t, in feet.	L_u, in feet.	2ϕ, in degrees and minutes.	W, at 160 lb. per cu. ft.	$y_a - y_w$, in feet.	y_a, in feet.	y_w, in feet.	M_p, in foot-pounds.	M_w, in foot-pounds.	$M = -M_p + M_w$.	A, in square feet.	I, in feet^4.
	(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
									−	+	−		
710	....	12.00	...	.....	.....	.....	.....	.....	.....	.....	.....	11.89	143
685	25	16.00	900	76–22	31 750	2.66	7.97	5.31	0.163 by 10^6	0.084 by 10^6	0.079 by 10^6	15.83	339
660	50	29.78	780	66–20	117 500	4.93	14 77	9.84	1.303 by 10^6	0.579 by 10^6	0.724 by 10^6	29.05	2 146
610	100	62.71	620	52–35	482 500	10.27	30.84	20.57	10.420 by 10^6	4.955 by 10^6	5.465 by 10^6	59.79	19 578
560	150	95.20	480	40–47	1 090 000	15.44	46.40	30.96	35.168 by 10^6	16.829 by 10^6	18.339 by 10^6	88.48	66 647
510	200	125.69	330	28–00	1 890 000	20.19	60.70	40.51	83.360 by 10^6	38.159 by 10^6	45.201 by 10^6	113.98	149 515
460	250	157.18	190	16–10	2 900 000	24.95	75.14	50.19	162.813 by 10^6	72.355 by 10^6	90.458 by 10^6	138.87	284 232
437.5	272.5	171.35	150	12–50	3 418 000	27.02	81.53	54.51	210.847 by 10^6	92.354 by 10^6	118.493 by 10^6	149.59	363 407

Elevation, in feet.	S_u, in feet^3.	S_d in feet^3.	K_u, in feet.	K_d, in feet.	$\frac{W}{A}$.	$\frac{M}{S_u}$.	$\frac{M}{S_d}$.	f_u, in pounds per square foot.	f_d, in pounds per square foot.	f_u, in pounds per square inch.	f_d, in pounds per square inch.	R_m, in feet.	$\frac{t}{R_m}$.
	(14)	(15)	(16)	(17)	(18)	(19)	(20)	(21)	(22)	(23)	(24)	(25)	(26)
						−	+	−	+	−	+		
710	.....	.....	.....	.....	... ..	.....	.....	.....	.....	.....	.....	669.0	.....
685	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	667.0	0.024
660	145.30	142.98	5.00	4.92	4 044	4 977	5 058	933	9 102	6.50	63.20	660.11	0.045
610	634.83	614.31	10.62	10.27	8 070	8 608	8 896	538	16 966	3.74	111.80	642.65	0.093
560	1 436.36	1 365.72	16.23	15.44	12 319	12 767	13 427	448	25 746	3.11	178.70	627.40	0.152
510	2 463.18	2 300.58	21.61	20.18	16 582	18 352	19 648	1 770	36 230	12.30	252.00	612.15	0.205
460	3 782.70	3 464.56	27.24	24.95	20 883	23 913	26 109	3 030	46 992	21.10	326.50	596.41	0.263
437.5	4 457.34	4 045.95	29.80	27.05	22 849	26 584	29 287	3 735	52 136	26.00	362.00	589.32	0.291

TABLE 1.—(*Continued.*)

Elevation, in feet.	C. (27)	P', in pounds. (28)	$\frac{R_u}{R_m}$. (29)	P, in pounds. (30)	$\delta_A' = C \times \frac{P R_m^2}{E t}$. (31)	δ_A', in inches. (32)	$\frac{M}{I}$. (33)	$\Sigma \frac{M}{I}$. (34)	δ_B', in inches. (35)	x. (36)	$H \times x$. (37)	$\frac{x^2}{2}$. (38)	δ_S', in inches. (39)
710	.....	.....	.....	.....	.....	.....	.....	72 236	0.410	272.50	74 256	37 128	0.159
685	1.852	1 563	1.01	1 578	0.2260	2.710	233.20	.....	0.345	.....	.....	.....	.....
660	1.740	3 125	1.021	3 192	0.2255	2.710	387.20	65 060	0.273	222.50	60 631	24 753	0.155
610	1.025	6 250	1.050	6 565	0.1232	1.478	279.12	51 160	0.177	172.50	47 006	14 878	0.138
560	0.305	9 375	1.076	10 100	0.0354	0.424	275.15	37 290	0.100	122.50	33 381	7 503	0.112
510	0.110	12 500	1.103	13 800	0.0126	0.151	302.32	22 860	0.053	72.50	19 756	2 628	0.074
460	.....	15 625	1.132	.....	0.000075*	0.0009	320.51	7 270	0.010	22.50	6 131	253	0.053
437.5	.....	17 030	1.146	.....	.....	.....	326.06	.....	.....	.....	.....	.....	.....

Elevation, in feet.	$\delta_B' + \delta_S'$ in inches. (40)	P, in pounds. (41)	S, in pounds. (42)	$\frac{S}{A}$. (43)	M_P', in foot-pounds. (44)	$M' = -M_P' + M_w$. (45)	$\frac{M'}{I}$. (46)	$\Sigma \frac{M'}{I}$. (47)	δ_B, in inches. (48)	δ_S, in inches. (49)	$\delta_C = \delta_B + \delta_S$, in inches. (50)	P_C, in pounds. (51)	P_A, in pounds. (52)
710	0.569	+ 2 630	.....	.....	.....	.....	.....	37 500	0.2750	0.1346	0.4096	.....	.....
685	0.502	— 13 800	— 11 170	706	± 0.00 by 10^6	0.0844 by 10^6	+249.00	40 500	0.2800	0.1833	0.3633	1 370	193
660	0.428	— 58 900	— 70 070	2 410	— 1.00 by 10^6	— 0.4208 by 10^6	—195 60	43 500	0.1900	0.1273	0.3173	2 750	375
610	0.315	—196 300	— 266 370	4 460	— 9.00 by 10^6	— 4.045 by 10^6	—207 00	31 220	0.1200	0.1130	0.2330	5 100	1 150
560	0.212	— 293 000	— 559 370	6 320	— 29.00 by 10^6	—12.171 by 10^6	—182.50	21 485	0.0600	0.0898	0.1498	6 600	2 775
510	0.127	—353 000	— 912 370	8 010	— 65.00 by 10^6	—26.841 by 10^6	—179.50	12 435	0.0250	0.0585	0.0835	7 500	5 000
460	0.068	—430 000	—1 342 370	9 640	—120.00 by 10^6	—47.645 by 10^6	—167.60	3 760	0.0050	0.0201	0.0251	10 400	5 225
437.5	.....	—281 200	—1 623 570	10 860	—153.00 by 10^6	—60.646 by 10^6	—167.00	.....	.....	.....	.....	17 030	.....

* Arch deflection similar to beam deflection.

TABLE 1.—(*Continued.*)

Elevation, in feet.	$P = P_G + P_A$, in pounds. (53)	$K = \frac{P_A}{P}$. (54)	$\delta A = K \times \delta A'$. (55)	$\frac{W}{A}$. (56)	$\frac{M'}{S'_u}$. (57)	$\frac{M'}{S'_d}$. (58)	f_u, in pounds per square foot. (59)	f_d, in pounds per square foot. (60)	f_u, in pounds per square inch. (61)	f_d, in pounds per square inch. (62)	Average arch thrust in pounds. (63)	Average arch stress, in pounds per square foot. (64)	f_A, in pounds per square inch. (65)
710	.....	.....	0.4006	.....	.....	.....	+	+	+	+	.....	.....	.....
685	1 563	.....	0.3360	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
660	3 125	0.124	0.3250	4 044	— 2 892	2 940	1 152	6 984	7.99	48.60	240 000	8 060	56.00
610	6 250	0.120	0.3250	3 070	— 6 950	7 180	1 120	15 250	7.77	105.70	450 000	7 180	49.80
560	9 375	0.134	0.2720	12 319	— 8 480	8 910	3 859	21 229	26.66	147.50	450 000	4 727	32.80
510	12 500	0.421	0.1785	16 582	—10 900	11 660	5 682	28 242	39.48	196.30	450 000*	3 580	24.90
460	15 625	0.400	0.0610	20 863	—12 580	13 790	8 308	34 613	57.70	240.50	400 000*	2 550	17.70
437.5	17 080	0.385	0.0300	22 849	—13 600	14 980	9 219	37 829	64.20	262.80	.....	.....	.....

Elevation, in feet.	A_A, in square feet. (66)	ΔA_A, in square feet. (67)	Percentage of A_A required. (68)	$\frac{M_0 = S_u \times W}{A}$. (69)	$M_P' = -M_0 - M_w$. (70)	h^2. (71)	x, in feet. (72)	$(h-x)$ in feet. (73)	Z †, in pounds. (74)	$M_P' = \frac{143}{160} \times M_P'$. (75)	x, in feet. (76)	$(h-x)$ in feet. (77)	Zt, in pounds. (78)
710	12.00	....	....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....
660	29.78	1.12	3.76	587 702	— 1 166 977	2 500	44.72	5.28	330	1 040 000	39.90	10.10	630
610	62.71	2.10	3.35	5 123 021	— 10 078 296	10 000	96.55	3.45	216	9 050 000	86.60	18.40	856
560	95.20	2.10	2.21	17 694 763	— 34 524 363	22 500	147.00	3.00	188	30 820 000	131.50	19.50	1 220
510	125.69	2.10	1.67	40 844 105	— 79 003 245	40 000	189.21	10.79	798	70 440 000	168.80	31.20	1 950
460	157.18	1.86	1.19	78 995 189	—151 850 189	62 500	231.99	18.01	1 125	135 400 000	207.80	42.20	2 630
437.5	171.35	....	....	101 846 297	—194 200 657	74 256	250.54	21.96	1 373	173 850 000	224.20	48.80	3 020

* Beam action is greater than arch action.
† Water load for $w = 160$.
‡ Water load for $w = 143$.

gravity, and the effective moments, M, are listed in Columns (5), (6), (7), and (11).

In Columns (12) to (24), Table 1, the areas, A, moments of inertia, I, section moduli, S_u and S_d, kern distance, K_u and K_d (Fig. 9(b)), and stresses, f_u and f_d (up-stream and down-stream face), are shown. In Columns (32), (35), and (39), the arch deflection, δ'_A, and the cantilever deflections, δ'_B and δ'_S, for full load (water surface at Elevation 710) are as shown in Fig. 9(a). The cantilever deflections, $\delta_C = \delta_B + \delta_S$, for partial load and the arch deflections, δ_A, for partial load are shown in Columns (50) and (55) to coincide closely enough to confirm the load distribution as shown in Fig. 9(e). A further adjustment of these deflections would change the respective loads very little. The stresses f_A, f_u, and f_d, due to partial arch and cantilever loads, respectively (Fig. 9(e)), are given in Columns (65), (59), (60), (61), and (62).

In determining the average arch thrust, the influence of elastic deformations has been included, and the result as shown in Columns (56) to (68), Table 1, indicates that, in the lower part of the dam, arch action is gradually being replaced by beam action until a condition is reached at the bottom of the dam where bending becomes negligible as compared with radial shear; that is, where the respective part of the dam acts as a wedge. Assuming the crushing strength of the concrete to be 1 500 lb. per sq. in., or 216 000 lb. per sq. ft., the area required to transmit this thrust is shown under δA_A, Column (67), and the maximum percentage of the existing area is shown in Column (68). Inasmuch as the behavior of an arch, as well as that of a beam of such dimensions as in the lower part of the dam, is of considerable uncertainty, it may be stated that the respective values for the percentage of A_A required, are meant to be indicative rather than exact.

In Columns (69) to (78), Table 1, the load limit lines are determined as shown in Fig. 9(e). They were calculated for $W = 160$ lb. per cu. ft. and $W = 143$ lb. per cu. ft., respectively. They represent the water load that may be brought against the dam so as to produce a stress, $f_u = \pm 0$, at the up-stream face.

The conditional equation, therefore, is:

$$f_u = \pm 0 = \frac{W}{A} - \frac{M_0}{S_u} \qquad (30)$$

$$M_0 = \frac{W \times S_u}{A} \qquad (31)$$

and as the sum of the moments must be equal to zero,

$$-M_P' + M_W + M_0 = 0$$

and,

$$-M_P' = -M_W - M_0$$

From Fig. 10:

$$M_P' = \frac{62.5\,h^2}{6} \times x$$

and,

$$x = \frac{6\,M_P'}{62.5\,h^2} = \frac{M_P'}{10.42\,h^2} \qquad (32)$$

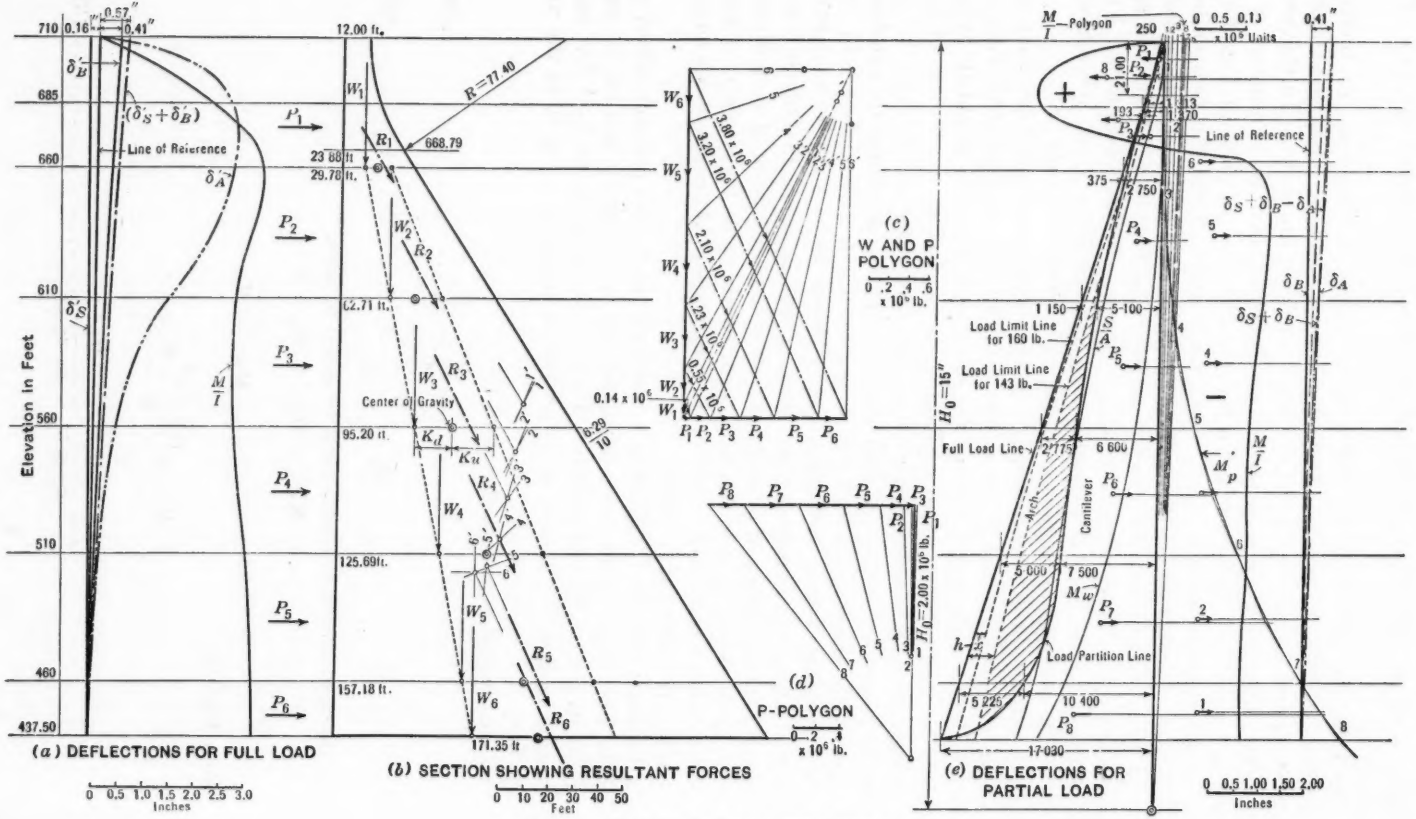

FIG. 9.—DESIGN OF EXCHEQUER DAM.

The cross-hatched area in Fig. 9(*e*) shows the margin between the load limit line for $W = 143$ lb. per cu. ft. and the load partition line, and may be considered as the "clearance of arch action" in this part of the dam.

Finally, the water loads, P, as shown in Fig. 9(*e*), have been combined with the weights, W (based on 160 lb. per cu. ft.) in Fig. 9(*c*), and the respective resultants are shown in Fig. 9(*b*) relative to the centers of gravity and the kern points of horizontal sections.

The formulas, as given by the author, hold good for any central angle except the one for the moment of inertia which holds for a hollow sector (trapezoid), 1 ft. wide at the up-stream face. In order to enable the determination of stresses of a section between any two radial planes (for instance, two contraction joints), a precise formula for the moment of inertia was derived by the writer, as follows:

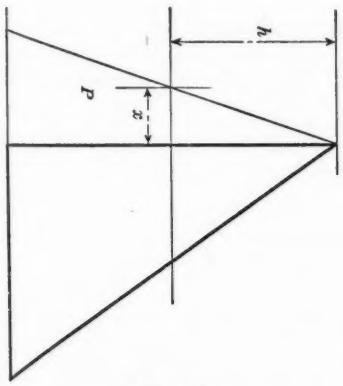

FIG. 10.

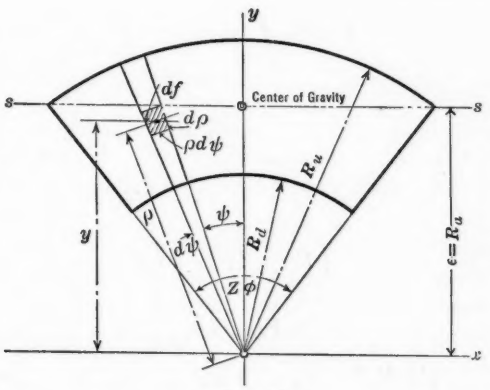

FIG. 11.

From Fig. 11:

$$I_x = 2\int_0^{\phi} y^2\, d\, f \qquad \ldots\ldots(33)$$

$$y = \rho \cos \psi$$

$$y^2 = \rho^2 \cos^2 \psi$$

$$d\, f = \rho\, d\, \rho\, d\, \psi$$

$$I_x = 2\int_0^{\phi}\int_{Rd}^{R_u} \rho^2 \cos^2 \psi\, d\, \psi\, \rho\, d\, \rho = 2\int_0^{\phi} \cos^2 \psi\, d\, \psi \int_{R_d}^{R_u} \rho^3\, d\, \rho$$

$$\int_0^{\phi} \cos^2 \psi\, d\, \psi = \left[\frac{\sin 2\phi}{4} + \frac{\phi}{2}\right] + C_1\,(= 0)$$

$$\int_{R_d}^{R_u} \rho^3\, d\, \rho = \left[\frac{\rho^4}{4}\right]_{R_d}^{R_u} + C_2\,(= 0) = \frac{R_u^4 - R_d^4}{4}$$

$$I_x = \frac{R_u^4 - R_d^4}{8}\,[\sin 2\phi + 2\,\phi] \qquad \ldots\ldots(34)$$

$$R_u^4 - R_d^4 = (R_u^2 - R_d^2)(R_u^2 + R_d^2) = (R_u - R_d)(R_u + R_d)$$
$$[(R_u + R_d)^2 - 2\,R_u\,R_d]$$
$$R_u - R_d = t$$
$$R_u + R_d = 2\,R_m$$
$$R_u^4 - R_d^4 = 2\,t\,R_m\,[4\,R_m^2 - 2\,R_u\,R_d]$$
$$I_x = \frac{t\,R_m}{4}\,[4\,R_m^2 - 2\,R_u\,R_d]\,[\sin 2\phi + 2\phi]$$
$$= t\,R_m\left[R_m^2 - \frac{R_u\,R_d}{2}\right][\sin 2\phi + 2\phi] \quad \ldots\ldots (35)$$

which is the equation for the moment of inertia about the x-axis. For the moment of inertia about the center of gravity of the section:

$$I_S = I_x - F\,\varepsilon^2 \quad \ldots\ldots (36)$$

in which,

$$F = 2\int_0^{\phi} d\,f = 2\int_0^{\phi}\int_{R_d}^{R_u} d\,\psi\,\rho\,d\,\rho = 2\,\phi\left[\frac{\rho^2}{2}\right]_{R_d}^{R_u}$$
$$= \phi\,[R_u^2 - R_d^2] = 2\,\phi\,t\,R_m \quad \ldots\ldots (37)$$
$$\varepsilon = R_d{}^* = \left(R_m + \frac{t^2}{12\,R_m}\right)\frac{\sin\,\phi}{\phi}$$
$$\varepsilon^2 = \left(R_m + \frac{t^2}{12\,R_m}\right)^2\frac{\sin^2\,\phi}{\phi^2}$$
$$\left(F \times \varepsilon^2\right) = 2\left(R_m + \frac{t^2}{12\,R_m}\right)^2\frac{\sin^2\,\phi}{\phi} \times t \times R_m$$
$$2\left(R_m + \frac{t^2}{12\,R_m}\right) = \frac{1}{72\,R_m^2}\left(12\,R_m^2 + t^2\right)$$
$$(12\,R_m^2 + t^2) = 3\,R_u^2 + 6\,R_u\,R_d + 3\,R_d^2 + R_u^2 - 2\,R_u\,R_d + R_d^2$$
$$= 4\,[R_u^2 + R_u\,R_d + R_d^2] = 4\,[(R_u + R_d)^2 - R_u\,R_d] = 4\,[4\,R_m^2 - R_u\,R_d]$$

Introducing this value:

$$\left(F \times \varepsilon^2\right) = \frac{16\,t}{72\,R_m}\left[4\,R_m^2 - R_u\,R_d\right]^2\frac{\sin^2\,\phi}{\phi}$$
$$= \frac{32 \times t}{9 \times R_m}\left[R_m^2 - \frac{R_u\,R_d}{4}\right]^2\frac{\sin^2\,\phi}{\phi} \quad \ldots\ldots (38)$$

Equation (36) now becomes:

$$I_S = t\,R_m\left[R_m^2 - \frac{R_u\,R_d}{2}\right]\left[\sin 2\,\phi + 2\phi\right]$$
$$- \frac{32 \times t}{9 \times R_m}\left[R_m^2 - \frac{R_u\,R_d}{4}\right]^2\frac{\sin^2\,\phi}{\phi} \quad \ldots\ldots (39)$$

WILLIAM CAIN,† M. AM. SOC. C. E. (by letter).‡—The writer has been much interested in this excellent paper. For the assumptions, all the derived formulas are exact. A volume 1 ft. wide at the up-stream face is considered.

* As derived by the author.

† Prof. Emeritus, Univ. of North Carolina, Chapel Hill, N. C.

‡ Received by the Secretary, October 22, 1927.

If this width had been taken as $d\,s$ in place of 1, then, at once, 1 could replace $\frac{\sin\phi}{\phi}$, and Equation (4)* could be more easily derived.

Curved gravity dams are ordinarily constructed with radial, vertical contraction joints. If these are not provided, the dam is likely to develop vertical cracks. Assuming such joints or cracks to extend from top to bottom, the author's theory applies to portions of the dam between cracks when the distance between them is not too large. If, however, by reinforcement or grouting, it is possible to realize a monolithic dam, then the theory presented does not apply and recourse must be had to the theory for the dam treated as a whole, as outlined by Fred A. Noetzli, M. Am. Soc. C. E.,† or an equivalent method may be used. However, if such dams are made by simply grouting the joints or cracks after full shrinkage has taken place, it is doubtful if the dam can be maintained as a monolith under extreme changes of temperature.

The author has treated completely the arched gravity dam of triangular cross-section with a vertical water face and has applied the results to a dam of such cross-section that, if the dam is first regarded as straight, with reservoir full, and uplift neglected, the stress at the heel is zero. Afterward, the dam of this same cross-section is considered curved with an up-stream radius of 674.7 ft.; the stress at the heel is again computed and found to be tension and, in amount, 25.9 lb. per sq. in.

The conclusion is inescapable that it is dangerous to neglect curvature in a curved gravity dam, and it is plain that the error increases as the radius diminishes. The error has been computed on the usual assumptions. There are likewise "uncertainties involved in determining the stresses near the foundation of a gravity dam", as the author notes.‡ It would seem, then, that the base of the dam should always be thicker than the usual middle-third theorem prescribes.

The author gives a complete solution for the arched gravity dam of a triangular cross-section with the up-stream face vertical. It may be of interest to indicate the solution for the usual cross-sections, as given in Fig. 12, where the lines, $F\,G$ and $G\,B$, are tangent to the curved down-stream profile, not drawn.

The value of M_c, the moment about C of the solid the cross-section of which is $A\,O\,B$ ($A\,O$ being vertical), is given directly by the author.§ For the remaining sections, Equation (7),

$$M_c = 2\,w \sin\phi \int r^2\,h\,d\,r$$

applies on substituting proper limits and values of r in terms of h.

Let the dotted lines in the sections, $E\,A\,O$, $O\,D\,F\,I$, and $I\,F\,G$, represent the variable ordinate, h, and let r be the radius, with center at C, corresponding to h, or the horizontal distance from C to the point where the dotted lines produced meet the base, $E\,B$. Also, let the horizontal radii from C to Points

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1137.
† *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 44.
‡ *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1142.
§ *Loc. cit.*, p. 1138.

E, A, and the points on $E\,B$ vertically under F and G, be designated by the symbols, r_e, r_a, r_f, and r_g, respectively. Then, from similar triangles,

Section $E\,A\,O$,

$$\frac{h}{O\,A} = \frac{r_e - r}{E\,A}$$

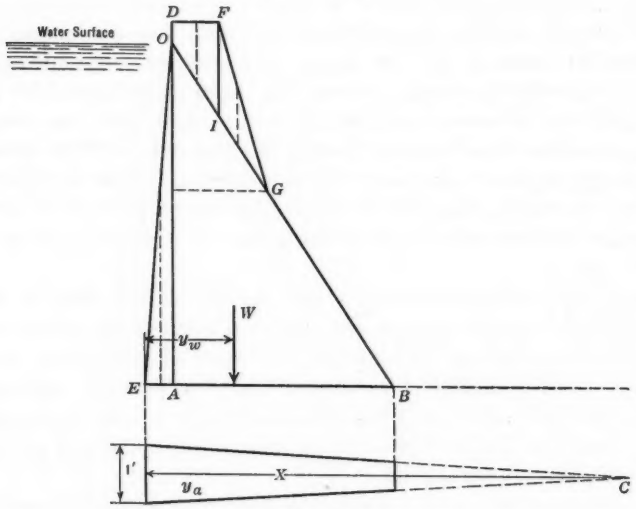

FIG. 12.

Section $O\,D\,F\,I$,

$$h = O\,D + (r_a - r)\tan\alpha$$

Section $I\,F\,G$,

$$\frac{h}{I\,F} = \frac{r - r_g}{r_f - r_g}$$

For a numerical illustration, the cross-section of the straight dam so thoroughly analyzed by the writer* will be adopted, and the dam will be regarded as curved, with an up-stream radius of 500 ft. The dimensions, in feet, will be given in reference to Fig. 12. The height to the water surface, $A\,O = 200$, $E\,A = 4$, and $A\,B = 130$, so that the base, $E\,B = 134$ ft. Also, $O\,D = 8$, $D\,F = 20$, and $I\,F = 38.8$; and, $\tan\alpha = \frac{200}{130} = \frac{100}{65}$. The radii are as follows:

$$r_e = 500;\ r_a = 496;\ r_f = 476;\ r_g = 463.5;\ r_b = 366$$

The radius to the vertical through the center of gravity of the section, $E\,O\,D\,F\,G\,B$, is 455.3 ft., and its area, 14 112 sq. ft.; so that the volume of the part of the curved dam having the central angle, $2\,\phi$, is,

$$14\,112 \times 2\,\phi \times 455.3 = 14\,112 \times \frac{1}{500} \times 455.3 = 12\,851 \text{ cu. ft.}$$

* "Stresses in Masonry Dams," *Transactions*, Am. Soc. C. E., Vol. LXIV (1909), p. 208. This paper is likewise given in the writer's "Retaining Walls" (Van Nostrand's Science Series No. 3, 7th Edition).

and its weight,

$$W = 12\,851\ w$$

Substituting the numerical values in the formulas just given, involving h and r, and solving for r:

For Section $E\ A\ O$,

$$r = 500 - 0.02\ h;\ dr = -0.02\ dh$$

For Section $O\ D\ F\ I$,

$$r = 501 - 0.65\ h;\ dr = -0.65\ dh$$

For Section $F\ I\ G$,

$$r = 463.5 + 0.322\ h;\ dr = 0.322\ dh$$

Substituting these values for the sections, $E\ A\ O$, $O\ D\ F\ I$, and $F\ I\ G$, in Equation (7), as quoted,

For Section $E\ A\ O$,

$$M_c = \frac{0.02}{500}\ w \int_{200}^{0} (500 - 0.02\ h)^2\ h\ d\ h = 197\,900\ w$$

For Section $O\ D\ F\ I$,

$$M_c = \frac{0.13}{100}\ w \int_{38.8}^{8} (501 - 0.65\ h)^2\ h\ d\ h = 218\,690\ w$$

For Section $F\ I\ G$,

$$M_c = \frac{0.0644}{100} \int_{0}^{38.8} (463.5 + 0.322\ h)^2\ h\ d\ h = 107\,820\ w$$

The M_c for the section, $A\ O\ B$, is determined directly from the formula given by the author,* in which, $H = 200$, $B = 130$, and $R_d = 366$, and is found to be, $M_c = 5\,352\,000\ w$. The total M_c, or moment about C for the entire solid, is the sum of the separate values. Therefore, $M_c = 5\,876\,410\ w$.

For a high dam, the parts outside $A\ O\ B$ are relatively small and a fair approximation to M_c for either $E\ A\ O$, $O\ D\ F\ I$, or $F\ I\ G$, is found by multiplying its area by the radius to the vertical through its center of gravity. This assumes a rectangular in place of a trapezoidal base for such parts. For this dam, the approximation was sufficiently close.

Having found M_c and W for the entire solid of the angle, 2ϕ, the radius to its center of gravity is,

$$\frac{M_c}{W} = \frac{5\,876\,410}{12\,851} = 457.27 \text{ ft.}$$

whence, the horizontal distance from this center of gravity to E is,

$$y_w = 500 - 457.27 = 42.73 \text{ ft.}$$

y_a, the distance from E to the center of gravity of the trapezoidal base, is given by Equation (4).† Thus, $y_a = 63.55$ ft. and $y_a - y_w = 20.82$ ft.

The remainder of the solution is effected by the author's Equations (10), (12), and (14).‡

$$x = 0.732;\ A = 116;\ I = 172\,400;\ M = -41\,266\,000$$

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1138.
† *Loc. cit.*, p. 1137.
‡ *Loc. cit.*, p. 1139.

$$w \text{ (assumed)} = \frac{5}{2\,(62.5)} \therefore W = 2\,008\,000 \text{ lb.}$$

$$s_u = \frac{W}{A} + \frac{M}{I}\, y_a = 17\,300 - 15\,240 = 2\,060 \text{ lb. per sq. ft. stress at } E$$

or 14.3 lb. per sq. in. compression at the up-stream edge under full water load. For the straight dam, the unit compression at E was 32.24 lb. per sq. in. This ignores uplift. Where that is to be considered, Equation (17)* directly applies.

The change in the vertical component of the stress at E from 32.24 lb. per sq. in. compression for the straight dam to 14.3 lb. per sq. in. for the curved dam, indicates that if the straight dam had been designed for such a value of w as to give zero stress at E, then the curved dam would be in tension at the heel, E. This result is exactly in line with the author's conclusion, and indicates that, for a curved gravity dam, the curvature should be considered in the analysis.

F. W. HANNA,† M. AM. SOC. C. E. (by letter).‡—The writer has been greatly interested in the author's discussion of arched gravity dams and believes that he has done the Engineering Profession a distinct service in presenting this subject in a comprehensive manner at this time. The author states that it is the purpose of his paper to show that it is not safe to neglect the curvature in calculating the stresses in an arched gravity dam and that this may lead to, and has led to, dams having tension in the up-stream face. He also states that the Habra Dam, in Algiers, and the Bouzey Dam, in France, failed from tension in the up-stream face several years after they were built. It appears to the writer that these claims are too broad and possibly may be misleading.

In regard to the Habra Dam, Wegmann states§ that the main dam was straight in plan (not curved), that the failure of the dam cannot be attributed to any defect in design, but was caused, in all probability, by faults in the execution of the work, and that the dam failed on account of a defective foundation. It appears, moreover, from a discussion of the dam by M. Leon Ponchet that the foundation was unsatisfactory; that defective stone may have been used in the masonry; that the sand used, was not good; that clayey red earth was used instead of sand for the inner part of the dam; that the hydraulic lime used was not very good; and that a combination of these circumstances caused the destruction of the dam. It appears, therefore, that this dam was not curved in plan and that its immediate failure was caused by defective foundation, possibly augmented by low-grade masonry in the body of the dam.

Wegmann also states that the Bouzey Dam was built straight in plan (not curved); that it was founded on red sandstone which was fissured and quite permeable; that considerable difficulty was experienced in the foundation trench from springs; that when the water reached a level of 33 ft. below

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1140.
† Hydr. Engr., East Bay Municipal Utility Dist., Oakland, Calif.
‡ Received by the Secretary, November 7, 1927.
§ "The Design and Construction of Dams."

the top of the dam springs of about 2 cu. ft. per sec. appeared on the lower side of the wall; that when the water reached a level 10.5 ft. below the top of the dam a portion of the dam, 444 ft. long, was shoved forward so as to form a curve convex down stream, causing additional fissures and increasing the flow of the springs to about 8 cu. ft. per sec.; that the failure of the dam is supposed to have been due to a greater tension at the up-stream face than the masonry could resist; and that this tension was probably increased by an upward water pressure under the dam. It appears from this statement that the initial failure of the Bouzey Dam was caused by sliding on the foundation, and that the final failure was probably caused by combined hydrostatic pressure, sliding, and shearing within the body of the dam due to poor masonry, and possibly injury to the dam through the initial failure. Here, as in the case of the Habra Dam, there was no curvature in plan, and neglect of the radial plane stress theory had nothing to do with the failure of the dam.

In order to prove the contention that it is never safe to neglect the curvature of a gravity dam in calculating the stresses, the author selects for analysis a radial-sided slice with a chord thickness of 1 ft. at the up-stream face of an arched gravity dam. The selection of this narrow slice is evidently a remnant of the custom of considering vertical slices of this thickness in calculating the stresses in straight gravity dams. The selection of a slice 1 ft. thick ignores the fact that in straight dams the analysis of a slice of this thickness will show the same stability and stress as a slice of any other thickness and that in curved dams the analysis of a radial slice of up-stream chord thickness of 1 ft. will show results materially different from those obtained by taking a radial slice of a greater thickness. Inasmuch as a 1-ft. radial slice cannot conceivably shear out from the surrounding material, an analysis for curved dams based on this assumption cannot yield reliable results. In view of this fact it would appear that stress computations in a curved gravity dam should be considered from other viewpoints.

In the usual dam site the radial plane analysis of arched gravity dams does not give consideration to the weight of all the material in the dam. In this connection, let Fig. 13 represent the plan and crown section of a symmetrical arched gravity dam placed in a box-shaped canyon. Let the dam be divided by vertical planes parallel to the axis of the canyon as shown in the diagram. The water pressure on each of these slices is made up of a transverse component equal to the water pressure times the slice thickness times the tangent of the angle of curvature of the dam from the center line of the canyon and a down-stream component equal to the slice thickness times the water pressure per unit width of the canyon. It is evident that the transverse components on the two sides of the canyon will balance one another and will have no effect on the overturning moment in a down-stream direction. It is also evident that the down-stream components are adequately resisted at the crown section and are still more adequately resisted for the oblique sections near the sides of the canyon, if the section of the dam is safe for a dam that is straight in plan. The parallel-sided sections can be made actual sections by placing all the contraction and construction joints parallel to the axis of the canyon. By referring to Fig. 13, it will be noted that the radial plane

theory entirely neglects the resistance of the triangles, *abc* and *def*, and that this material may be omitted from the dam since no arching action is to be relied on in this theory.

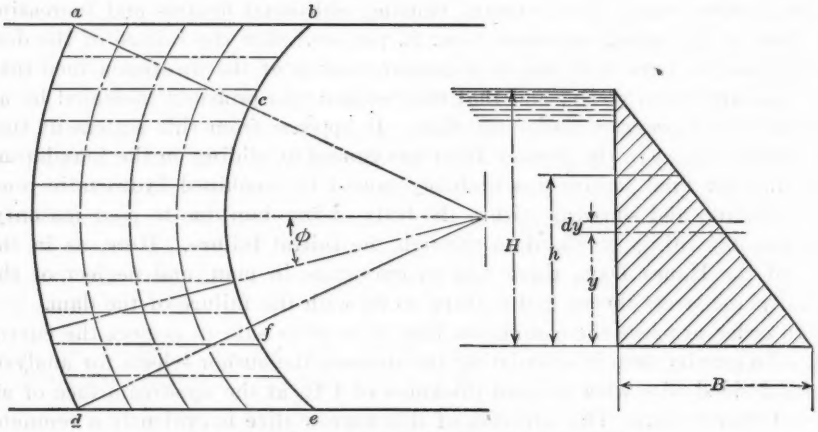

FIG. 13.—SYMMETRICAL ARCHED GRAVITY DAM.

In an arched gravity dam with properly spaced, constructed, and grouted contraction joints, the dam can generally be relied on to act as a unit. This fact is entirely ignored in the radial plane analysis theory. It brings arch action and increased moment resistance into play. Let Fig. 14 show the plan and section of a triangular dam; and, in order to simplify the mathe-

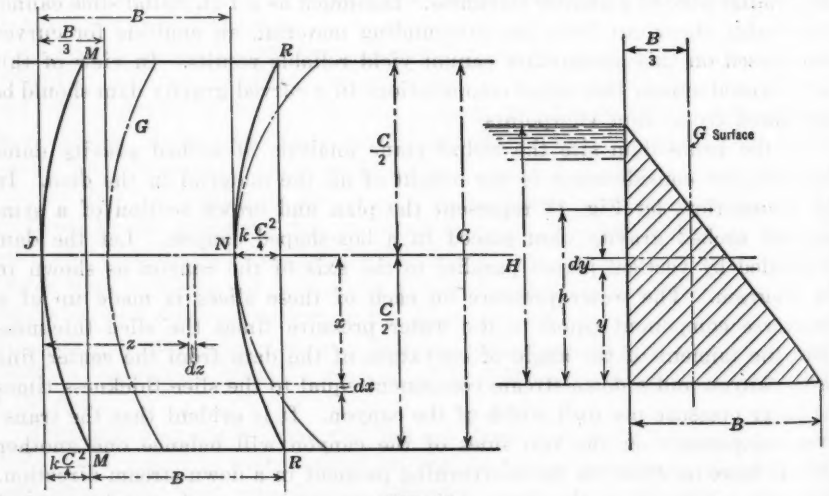

FIG. 14.

matical treatment, let like parabolic curves bound the up-stream and down-stream faces of the base. It will be possible to substitute such parabolic curves for the usual arcs without material error. All vertical sections taken parallel to the crown will be equal, and the total weight of material in the dam

will be the same as that in a straight dam of the same section. The basis of comparison will be very slightly unfair to the curved dam of uniform radial section. Let the co-ordinate axes be taken as coincident with the axis of the canyon and a line perpendicular thereto at the up-stream edge, J. Using the notation shown in Fig. 14, let the equation of the parabola representing the up-stream edge of the base be:

$$z = k\,x^2 \ldots\ldots (40)$$

that representing the line, G, at one-third the distance between the heel and toe on a line parallel to the z-axis be:

$$z - \frac{B}{3} = k\,x^2 \ldots\ldots (41)$$

and that representing the down-stream edge of the base:

$$z - B = k\,x^2 \ldots\ldots (42)$$

As already noted, the weight, W, of the dam will be the same as that for a straight dam, or,

$$W = \tfrac{1}{2}\,B\,H\,c\,w \ldots\ldots (43)$$

in which, w is the weight of a cubic unit of the masonry. The moment, M, of this weight about the heel, J, is expressed by the following equation:

$$M = \int_{-\frac{c}{2}}^{\frac{c}{2}} \frac{w\,B\,H}{2}\,z'\,dx \ldots\ldots (44)$$

in which, $z' = \frac{B}{3} + k\,x^2$, and which, on integration for the limits shown, becomes,

$$M = \frac{w\,B\,H\,c}{6}\left(B + \frac{k\,c^2}{4}\right) \ldots\ldots (45)$$

Let the intensity of the hydrostatic uplift on the base be $62.5\,f\,H$ at the heel and zero at the toe, varying linearly between these points. The total uplift will then be $31.25\,f\,B\,H\,c$ as for a straight dam. For every slice of the dam the center of gravity of uplift may be taken vertical below the center of gravity of the weight. The moment, M_2 of the uplift about the heel, J, derived similarly to Equation (45), becomes,

$$M_2 = 10.42\,f\,B\,H\,c\left(B\,\frac{k\,c^2}{4}\right) \ldots\ldots (46)$$

Also, as in the case of a straight dam, the horizontal water pressure, P, may be expressed as follows:

$$P = 31.25\,f\,c\,H^2 \ldots\ldots (47)$$

and the moment thereof about the heel, as,

$$M_3 = 10.42\,f\,c\,H^3 \ldots\ldots (48)$$

Let it be assumed that the vertical unit pressure under the base varies in a line parallel to the z-axis uniformly from D at the points, R, farthest down stream to U at the point, J, farthest up stream; and let Q represent the value of this pressure at any distance, z, from the x-axis. When $x = 0$, $z = 0$ from

Equation (40) and B from Equation (42) for the up-stream edge at the point, J, and for the down-stream edge, at the point, N, respectively; and when $x = \frac{c}{2}$, $z = \frac{k\,c^2}{4}$ for the up-stream edge at the point, M, and $B + \frac{k\,c^2}{4}$ for the down-stream edge of the base at the points, R. For any intermediate point at the distance, z, from the x-axis,

$$Q = U + \left(\frac{z}{B + \frac{k\,c^2}{4}}\right)(D - U) \quad \ldots\ldots\ldots\ldots\ldots\ldots (49)$$

The total base pressure, P, is that on the area, $M\,J\,M\,R\,N\,R$, which may be expressed as follows:

$$P = 2\int_0^{\frac{c}{2}} \int_{k\,x^2}^{(B + k\,x^2)} \left[U + \left(\frac{z}{B + \frac{k\,c^2}{4}}\right)(D - U)\right] d\,z\,d\,x \ldots\ldots (50)$$

which, on integration, equals,

$$P = B\,U\,c + \frac{\left(\frac{B^2\,c}{2} + \frac{B\,k\,c^3}{12}\right)(D - U)}{\left(B + \frac{k\,c^2}{4}\right)} \quad \ldots\ldots\ldots\ldots\ldots\ldots (51)$$

In a similar manner the moment, M_4, of the base pressure about the heel at J is,

$$M_4 = \int_0^{\frac{c}{2}} \int_{k\,x^2}^{B + k\,x^2} \left[U + \left(\frac{D - U}{B + \frac{k\,c^2}{4}}\right) z\right] z\,d\,z\,d\,x \ldots\ldots\ldots (52)$$

which, on integration, equals,

$$M_4 = c\,B\left[\left(\frac{B}{2} + \frac{k\,c^2}{12}\right) U + \left(\frac{D - U}{B + \frac{k\,c^2}{4}}\right)\left(\frac{B^2}{3} + \frac{B\,k\,c^2}{12} + \frac{k^2\,c^4}{80}\right)\right] \ldots (53)$$

Let consideration now be given to results obtained from applying Equations (51) and (53), involving the parabolically curved dam with stresses calculated on a parallel-sided vertical slice, 100 ft. wide, as compared to a vertical slice, 100 ft. wide, of a straight gravity dam of the same section. Take a horizontal section of the Exchequer Dam at Elevation 427.5 as cited by the author,* in which, $H = 272.5$ ft. and $B = 171.4$ ft., and let $w = 160$ lb. per cu. ft. For a straight dam of this section and masonry weight, the stress at the heel is zero. For a parabolically curved dam, a solution of Equations (51) and (53) gives U equal to 2.6 lb. per sq. in. compression at the heel, which is negligibly different from the zero stress obtained for the straight dam. This shows that, if stresses in an arched gravity dam are calculated on parallel-sided sections of reasonable width, the results do not differ materially from those obtained from calculating on the basis of the usual methods for straight gravity dams.

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1141.

In the matter of arch action coming into play to relieve the overturning moment from water pressure in an arched gravity dam, let the Exchequer Dam again be considered, using the triangular vertical up-stream face section assumed by the author; and let the water elevation be assumed to be at the crest of the triangle section. When any gravity dam is subjected to water pressure, as here assumed, the down-stream fibers of the dam are compressed by bending moment much more than the up-stream fibers, which causes the dam to be deflected down stream considerably. Shearing action also tends to increase this down-stream deflection. Computations of these combined deflections for the Exchequer Dam show the total deflection at 50 ft. below the crest of the dam on the axis thereof to be about 0.74 in.; at 100 ft. below the top of the dam, 0.51 in.; and at 150 ft. below the top of the dam, 0.32 in. Assuming the water load to be carried entirely by the arch and computing the crown deflections at the same three elevations, the corresponding deflections are found to be 3.18 in., 2.28 in., and 0.81 in., respectively. These figures indicate that the cantilever deflections from the water load in the gravity section at various levels are a very considerable proportion of those of the arch sections, indicating that an appreciable proportion of the water pressure will be taken by arch action in this dam, much more, in fact, than any deficiency of computed stress resulting from applying parallel-sided instead of radial-sided slices of the dam.

The arch deflections given in the preceding paragraph are computed with Cain's formula for fixed-ended thin circular arches under normal loads. The deflections of the gravity section are computed by means of the law of displacements of the internal work theory, the fundamental formula for which is as follows:

$$\Delta_h = \int_0^h \frac{M\,m}{E\,I}\,d\,y + 3\int_0^h \frac{V\,v}{E\,A}\,d\,y \ldots\ldots\ldots\ldots\ldots\ldots (54)$$

in which, M and V are the moment and shear due to the water load acting in producing stress on the elementary strip, dy, while m and v are the respective moment and shear on dy for a unit load of 1 lb. applied at the point of deflection, h, all as shown in Fig. 13. By assigning proper values to the various quantities involved in the fundamental formula and integrating, Δ_h, for any point, h, is found to be,

$$\Delta_h = \frac{62.5\,H^3\,h^2}{E\,B^3} + \frac{46.875\,H}{E\,B}\,(2\,H\,h - h^2) \ldots\ldots\ldots\ldots (55)$$

There are, however, cases where it may be desirable to compute the stresses in arched gravity dams on the basis of radial-sided slices, such as in a canyon converging down stream, with abutments not suitable for taking arch thrust, and such as may give better foundation conditions for a curved structure than for a straight structure combined with earth abutments or abutments otherwise not suited to taking the arch thrust. In such cases, it is desirable to compute the stresses on the theory of radial-sided slices. Formulas for considering stresses on this theory are not at all out of place, but they should not be allowed to displace the customary and reliable method of computing stresses

in arched gravity dams by the parallel-sided slice theory. In view of this statement the writer is submitting herein general formulas for this purpose, developed by means of the principles of calculus which check and supplement the formulas presented by the author.

Development of Formulas for Effect of Curvature on Stresses.—Assume a vertical slice of an arched gravity dam between radial planes thereof; let this vertical slice be divided into convenient segments by horizontal planes; and let Fig. 15 represent the plan and sectional elevation of one of these segments. Also, let the symbols shown on the diagram represent the quantities there indicated. Assume each segment to be acted upon only by water pressure and gravity, and take moments about a horizontal line perpendicular to the radial plane bisecting the segment and passing through the center of curvature called the center axis in the plane of the base of the segment. Consider an elementary volume and an elementary surface of the segment of the dam as shown in Fig. 15, and extend these considerations to the entire segment thereof by integration of the differential quantities between proper limits. Let all units be expressed in pounds and feet.

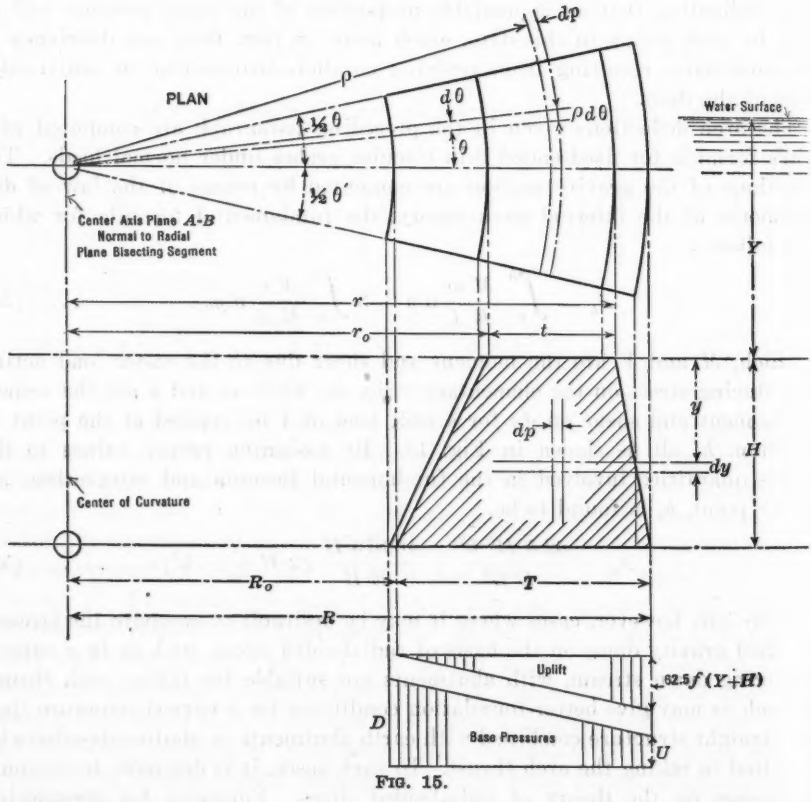

FIG. 15.

If w represents the weight per cubic foot of material in the segment, then the weight of the elementary volume, m, is $w\ dy\ d\rho\ \rho\ d\theta$, and the moment of

m about the center axis, AB, is $w\ dy\ d\rho\ \rho d\theta\ \rho \cos\theta$. The total weight of the segment and the total moment thereof about the center axis are readily found by integrating these differential quantities between the proper limits as follows:

$$W = w \int_{y=0}^{y=H} \int_{\rho = r_0 + \frac{y}{H}(R_0 - r_0)}^{\rho = r + \frac{y}{H}(R-r)} \int_{\theta=-\frac{\theta'}{2}}^{\theta=\frac{\theta'}{2}} d\,y\,d\,\rho\,\rho\,d\,\theta \ldots\ldots (56)$$

whence,

$$W = \frac{w\ H\ \theta'}{6}\left[\left(\frac{R^3 - r^3}{R-r}\right) - \left(\frac{R_0^{\,3} - R_0^{\,3}}{R_0 - r_0}\right)\right] \ldots\ldots\ldots\ldots (57)$$

and,

$$M = w \int_{y=0}^{y=H} \int_{\rho = r_0 + \frac{y}{H}(R_0 - r_0)}^{\rho = r + \frac{y}{H}(R-r)} \int_{\theta=-\frac{\theta'}{2}}^{\theta=\frac{\theta'}{2}} d\,y\,d\,\rho\,\rho\,d\,\theta\,\rho\cos\theta \ldots. (58)$$

whence,

$$M = \frac{w\ H \sin\frac{\theta}{2}}{6}\left[\left(\frac{R^4 - r^4}{R-r}\right) - \left(\frac{R_0^{\,4} - r_0^{\,4}}{R_0 - r_0}\right)\right] \ldots\ldots\ldots (59)$$

The up-stream surface of the segment is acted on both by vertical and horizontal water pressure. The vertical pressure acts in conjunction with the weight of the segment to increase its stability and the horizontal pressure acts in opposition thereto to produce overturning. In accordance with the principles of hydraulics, the vertical pressure acts on the horizontal projection of the surface of the segment and the horizontal pressure acts on the vertical projection of the surface.

The horizontal projection of the elementary area, n, of the up-stream face of the segment lies at a horizontal distance from the center axis of $r + \frac{y}{H}(R-r)$, and it has a width of $\frac{d\,y}{H}(R-r)$. Its area, therefore, is,

$$\left(\frac{R-r}{H}\right)\left[r + \frac{y}{H}(R-r)\right] d\,y\,d\,\theta$$

The vertical water pressure on this area is,

$$62.5\left(\frac{R-r}{H}\right)(Y+y)\left[r + \frac{y}{H}(R-r)\right] d\,y\,d\,\theta$$

and the moment of the vertical water pressure on this area about the center axis is,

$$62.5\left(\frac{R-r}{H}\right)(Y+y)\left[r + \frac{y}{H}(R-r)\right]^2 d\,y\cos\theta\,d\,\theta$$

The integration of these differential quantities gives the total vertical water pressure on the up-stream face of the segment and its total moment about the center axis. Let W_1 and M_1 represent the respective total pressure and moment; then,

$$W_1 = 62.5\left(\frac{R-r}{H}\right)\int_{y=0}^{y=H}\int_{\theta=-\frac{\theta'}{2}}^{\theta=\frac{\theta'}{2}}(Y+y)\left[r + \frac{y}{H}(R-r)\right] d\,y\,d\,\theta. (60)$$

whence,

$$W_1 = 10.42\,\theta'\,(R - r)\,[3\,Y\,(R + r) + H\,(2\,R + r)] \ldots\ldots\ldots (61)$$

and,

$$M_1 = 62.5\left(\frac{R - r}{H}\right)\int_{y=0}^{y=H}\int_{\theta=-\frac{\theta'}{2}}^{\theta=\frac{\theta'}{2}} (Y + y)\left[r + \frac{y}{H}(R - r)\right]^2 d\,y\cos\theta\,d\,\theta \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (62)$$

whence,

$$M_1 = 41.66\sin\frac{\theta'}{2}(R - r)\left[Y\left(\frac{R^3 - r^3}{R - r}\right) + \frac{1}{4}H\,(3\,R^2 + 2\,R\,r + r^2)\right]. (63)$$

In determining the horizontal water pressure acting on the up-stream face of the segment, the vertical projection of an elementary areal strip, q, across the up-stream face of the segment is used. The vertical projection of this area is,

$$2\left[r + \frac{y}{H}(R - r)\right]\sin\frac{\theta'}{2}\,d\,y$$

the horizontal water pressure on it is,

$$125\sin\frac{\theta'}{2}(Y + y)\left[r + \frac{y}{H}(R - r)\right]d\,y$$

and the moment of this pressure about the center axis is,

$$125\sin\frac{\theta'}{2}(Y + y)\left[r + \frac{y}{H}(R - r)\right](H - y)\,d\,y$$

By integrating these differential quantities and denoting W_2 and M_2, as the total pressure and moment, respectively, the following values are found:

$$W_2 = 125\sin\frac{\theta'}{2}\int_{y=0}^{y=H}\left[r + \frac{y}{H}(R - r)\right](Y + y)\,d\,y \ldots\ldots (64)$$

whence,

$$W_2 = 125\,H\sin\frac{\theta'}{2}\left[\frac{Y}{2}(R + r) + \frac{H}{3}\left(R + \frac{1}{2}r\right)\right] \ldots\ldots\ldots (65)$$

and,

$$M_2 = 125\sin\frac{\theta'}{2}\int_{y=0}^{y=H}\left[r + \frac{y}{H}(R - r)\right](Y + y)(H - y)\,d\,y \ldots (66)$$

whence,

$$M_2 = 10.42\,H^2\sin\frac{\theta'}{2}\left[2\,Y\,(R + 2\,r) + H\,(R + r)\right] \ldots\ldots\ldots (67)$$

The uplift pressure on the base of the dam varies with the permeability of its foundation, but it is believed to be a fair assumption for the case here under consideration to assume: (1) That the intensity of pressure at the heel of the dam is equal to some specified factor times the intensity of water pressure at that point; (2) that this intensity of pressure is zero at the toe of the dam; and (3) that the intensity of pressure varies as a straight line between the heel and toe. If P designates the entire uplift on the base of the dam

and f the coefficient of intensity of water pressure at the heel, and M_3 the moment of the uplift, P, on the base of the dam about the center axis, then,

$$P = \int_{\rho = R_0}^{\rho = R} \int_{\theta = -\frac{\theta'}{2}}^{\theta = \frac{\theta'}{2}} d\rho\, \rho\, d\theta\, 62.5\,(Y + H)\, f \left(\frac{\rho - R_0}{R - R_0}\right) \ldots\ldots (68)$$

whence,

$$P = f \left(\frac{62.5\,(Y + H)\,\theta'}{(R - R_0)}\right)\left[\frac{1}{3}\,(R^3 - {R_0}^3) - \frac{R_0}{2}\,(R^2 - {R_0}^2)\right] \ldots (69)$$

and,

$$M_3 = \int_{\rho = R_0}^{\rho = R} \int_{\theta = -\frac{\theta'}{2}}^{\theta = \frac{\theta'}{2}} d\rho\, \rho\, d\theta\, 62.5\,(Y + H)\, f \left(\frac{\rho - R_0}{R - R_0}\right) \rho \cos\theta \ldots (70)$$

whence,

$$M_3 = 125\, f \sin\frac{\theta'}{2}\,(Y + H)\left[\frac{1}{4}\left(\frac{R^4 - {R_0}^4}{R - R_0}\right) - \frac{R_0}{3}\left(\frac{R^3 - {R_0}^3}{R - R_0}\right)\right] \ldots (71)$$

The downward pressure on the foundation of the base of the dam is due to the weight of the superimposed masonry plus the vertical water pressure on the up-stream face thereof due to batter. The downward pressure at any intermediate point on an elementary area, $d\rho\ \rho d\theta$, is,

$$D - \frac{\left(\rho \cos\theta - R_0 \cos\frac{\theta'}{2}\right)}{R - R_0 \cos\frac{\theta'}{2}}\,(D - U)$$

in which, D is the intensity of pressure at the toe and U is the intensity of this pressure at the heel. Now, if P_1 equals the entire downward pressure on the foundation, and M_4 the moment thereof about the center axis,

$$P_1 = \int_{\rho = R_0}^{\rho = R} \int_{\theta = -\frac{\theta'}{2}}^{\theta = \frac{\theta'}{2}} d\rho\, \rho\, d\theta \left[D - \frac{\rho \cos\theta - R_0 \cos\frac{\theta'}{2}}{R - R_0 \cos\frac{\theta'}{2}}\,(D - U)\right] \ldots (72)$$

whence,

$$P_1 = D\,\theta'\left(\frac{R^2 - {R_0}^2}{2}\right) - (D - U)$$
$$\left[\frac{2 \sin\frac{\theta'}{2}\left(\frac{R^3 - {R_0}^3}{3}\right) - \theta'\, R_0 \cos\frac{\theta'}{2}\left(\frac{R^2 - {R_0}^2}{2}\right)}{R - R_0 \cos\frac{\theta'}{2}}\right] \ldots\ldots (73)$$

and,

$$M_4 = \int_{\rho = R_0}^{\rho = R} \int_{\theta = -\frac{\theta'}{2}}^{\theta = \frac{\theta'}{2}} d\rho\, \rho\, d\theta\, \rho \cos\theta$$
$$\left[D - \left(\frac{\rho \cos\theta - R_0 \cos\frac{\theta'}{2}}{R - R_0 \cos\frac{\theta'}{2}}\right)(D - U)\right] \ldots\ldots (74)$$

whence,

$$M_4 = D\frac{2}{3}\sin\frac{\theta}{2}(R^3 - R_0^3) - (D - U)$$

$$\left[\frac{\frac{1}{8}(\theta' + \sin\theta')(R^4 - R_0^4) - \frac{1}{3}R_0(\sin\theta')(R^3 - R_0^3)}{R - R_0\cos\frac{\theta'}{2}}\right] \quad \ldots\ldots (75)$$

The pressure intensities, D and U, can now be found from a combination of Equations (73) and (75), the total base pressure having been previously found from Equations (57) and (65), and the moments thereof from Equations (59), (63), and (67). Where hydrostatic uplift is assumed to act on the base of the dam, Equations (69) and (71) must be used to determine the modifications of stresses resulting therefrom.

Acknowledgment is made to T. L. E. Haug, Designing Engineer, East Bay Municipal Utility District, Oakland, Calif., for valuable assistance in developing and checking the methods of analysis and formulas herein presented.

E. W. KRAMER,* M. AM. SOC. C. E. (by letter).†—This able paper is very timely in calling attention to the danger of allowing tension in the heel of any dam that depends entirely on gravity action for its stability, and also in emphasizing the importance of considering a slice between two vertical radial planes in analyzing the stresses in a gravity dam arched in plan. The timeliness is due to the number of gravity dams exceeding 300 ft. in height, either proposed or under construction.

Among the important questions arising in connection with the planning of these structures, on which there seems to be some difference of opinion, are the following:

1.—Should a gravity dam be arched in plan where topographical conditions permit?

2.—If arched in plan, is a smaller safety factor with respect to maximum compression at the toe, tension at the heel, and uplift assumption permissible?

3.—What is the amount of uplift for which provision should be made?

4.—Should a section between radial or parallel vertical planes be considered in determining stresses?

5.—What should the allowable working stresses be?

These problems are, of course, not new. However, knowledge on the questions involved is continually being increased by observing dams actually constructed, as these subjects of observation are continually increasing in height and varying in form. Great advancement in the mathematical treatment of stresses and strains in structures has been made in recent years as well as an increase in knowledge as to the strength of the materials of which these structures are built. Therefore, it is hoped that the following discussion, in which an attempt is made to enlarge on some of the points made by Mr. Jakobsen, will be at least of passing interest.

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† Received by the Secretary November 16, 1927.

The answer to the first question depends to a great extent upon the answers to the other questions, and more particularly to the second question. There are, however, some points that can be considered independently.

As a general rule a larger concrete yardage will be required at a greater cost per yard for the gravity dam arched in plan than for the straight dam, regardless of the stand taken by the designer on the other questions. Therefore, the arched dam will cost more. An exception to this rule might occur in a case where the arching throws the center of the dam up stream on better and higher foundations.

Having decided that the arched plan will cost more, it is then necessary to justify this extra cost. The usual answer is that an additional safety factor is provided. It is the writer's opinion, however, that the arching affords only a secondary or delayed support in the case of dams arched on a greater radius than about 800 ft. This support may not come into play under certain very reasonable assumptions until after partial failure has occurred.

For instance, observations of the temperature of large concrete dams show that in the interior it rises rapidly as high as 110° Fahr. The gradual cooling will open the contraction joints. As this cooling covers a long period, there is no assurance that these contraction joints can be grouted before the load comes on the dam. In fact, it is not a general practice to grout the contraction joints of gravity dams. Some of the engineers most prominent in the design of high dams hold that it is inadvisable to do so. Observations show that these cracks are sometimes as large as $\frac{1}{10}$ in. For a dam of long radius, computations and observations indicate that the cantilever will not deflect sufficiently to close the cracks, which is essential to bring the arch into play.

An incipient failure caused by the dam sliding on its base a matter of some hundredths of a foot would bring the arch into play. It would also greatly increase the uplift. The support given by the arch might prevent a complete and rapid failure and a serious disaster involving loss of life and property, while it might not preserve the usefulness of the dam. This additional safety factor, no doubt, has at least psychological value.

On the other hand, the science of engineering has advanced in regard to methods of exploring foundations, grouting and draining foundations, knowledge of designing and placing concrete, designing cross-section, etc., to such a point that the question arises, "is not the additional safety provided by arching a gravity dam on a long radius in most cases superfluous?". Before the designer will answer this question, it is likely that he will first determine what the arching will cost. To do this it will be necessary for him to make a decision in regard to the other questions.

Confining the answer to gravity dams with radii of more than 700 or 800 ft., the writer's answer to the second question would be that it is not advisable to use a smaller safety factor with respect to compression at the toe, tension at the heel, or uplift assumption, than is customary in a straight dam. For radii of less than, say, 700 ft., the answer is not so evident.

To support this answer take the case of the Exchequer Dam of the Merced Irrigation District, Merced, Calif., referred to by Mr. Jakobsen.* The cross-section of this dam, as published,† is shown on Fig. 16. The radius of the up-stream face at the crest is 674 feet. The section is very nearly triangular. In order to simplify the deflection computations, assume the triangular cross-section that would be obtained by extending the principal batter of the down-stream face (10 on 6.29) to the crest and foundations. For a height of 320 ft. the bottom width will be 202 ft. and the top width zero (very nearly).

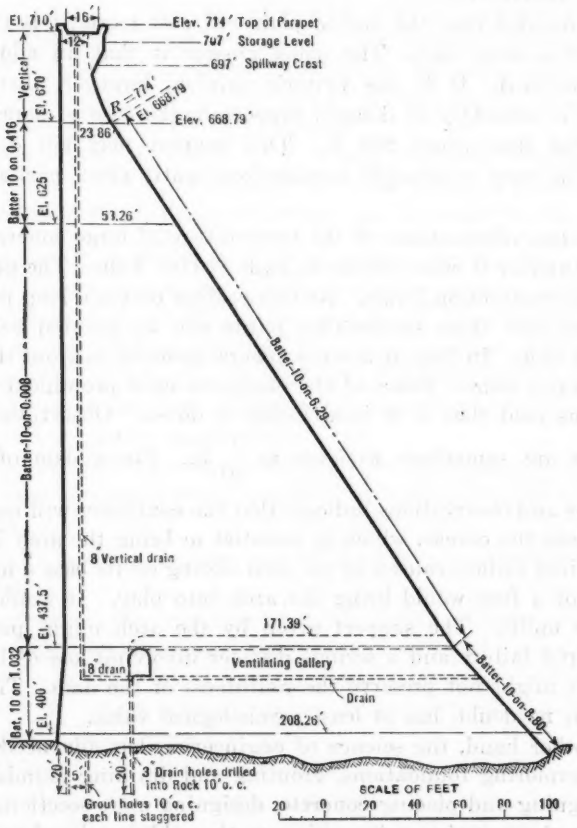

FIG. 16.—MAXIMUM CROSS-SECTION OF EXCHEQUER DAM.

The assumption is made that the cantilever carries the entire load and gives as great a deflection as can occur, unless cracks develop in the up-stream face. Therefore, for the present purpose, this assumption is allowable. The following equation‡ will be used for obtaining the cantilever deflection at the

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1141.

† *Western Construction News*, January 25, 1926.

‡ Derived by F. A. Noetzli, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 6.

crest, neglecting shear and yielding of foundations, and assuming that the cantilever takes the entire load:

$$Y_c = \frac{62.5\,H^3}{E\,t_b^3}\,X^2$$

in which,

Y_c = deflection, neglecting shear;
H = height of dam;
t_b = thickness of base; and
X = height of point above base for which deflection is to be determined.

For the Exchequer Dam, remembering that $t_b = 0.63\,H$,

$$Y_c = \frac{62.5\,H^3}{E\,(0.63\,H)^3}\,X^2 = 0.000000868\,X^2$$

Therefore, Y_c for the crest, when $X = 320$, is 0.0889 ft.

In his discussion of Mr. Noetzli's paper, to which reference has been made, Mr. Jakobsen shows that the deflection due to shear is very considerable in a case of this kind, and that it should be added to the computed cantilever deflection with shear neglected.

Mr. Jakobsen developed the equivalent of the following formula* for the determination of the cantilever deflection due to shear alone at the crest of the dam where $X = H$,

$$Y_s = \frac{1.2 \times 31.3 \times H^3}{144 \times G \times t_b}$$

Values of $E = 2\,000\,000$ and $G = \frac{2}{5}\,E$ are low and will give a high value for the deflections. They will be used, therefore, in order to get the largest probable deflection.

Taking $H = 320$ ft. and $t_b = 202$ ft. for the Exchequer Dam gives $Y_s = 0.0265$ ft. at the crest. Therefore, the total cantilever deflection,

$$Y_c + Y_s = 0.0889 + 0.0265 = 0.1154 \text{ ft.}$$

Consider a block of the dam between two vertical contraction joints 50 ft. apart on the up-stream face. For a fall of 20° Fahr. the contraction will be,

$$0.0000055 \times 20 \times 50 = 0.0055 \text{ ft.} = 0.076 \text{ in.}$$

This dimension is of the order of the size of the cracks in the Mulholland Dam referred to by Mr. Jakobsen.† To close these cracks on the up-stream face at the crest for a radius of 674 ft., using the relation between the arc and radius of a circle, will require a shortening of the radius and hence a deflection of,

$$Y = \frac{674}{50}\,(0.0055) = 0.0741 \text{ ft.}$$

Since this is considerably less than the cantilever deflection, 0.1154 ft., it would seem that the cracks would be closed before full cantilever deflection took place and that arch action would be developed in the upper part of the dam.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 100.
† *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1142.

This would materially lessen the cantilever stresses. Hence, the doubtful exception made by the writer for dams with radii less than 700 or 800 ft. The foregoing statement is based on the assumption that the maximum fall in temperature of the concrete below the temperature at which it took its permanent form will not greatly exceed 20° Fahr.

Suppose, however, that the radius of the up-stream face at the crest was 1 200 ft. To close the cracks at the up-stream face would require a shortening of the radius at the crest or, in other words, of the deflection at the crest of,

$$Y = \frac{1\,200}{50}\,(0.0055) = 0.132 \text{ ft.}$$

which is considerably greater than the cantilever deflection at the top of the dam where it would be greatest until the arch action came into play. Remember, also, that a most liberal estimate was made of the cantilever deflection and that shrinkage of the concrete, due to setting, was neglected in computing the width of the contraction crack.

The selection of a fall in temperature of 20° Fahr. may be open to criticism as being too small when compared with the high temperature that the concrete may reach while it is setting. On the other hand these high temperatures are not all closing temperatures. The practice of pouring the dam in alternate blocks allows some time factor between the peaks in the block temperatures. One alternate set of blocks may have passed their maximum temperature before the intermediate blocks were poured.

It is admitted that a greater drop of mean temperature below that at which the concrete took its permanent form in the case of the Exchequer Dam might be justified. On the other hand, it is believed that a drop of at least 20° occurred.

If any unreasonable assumptions appear in this argument, they were adverse to the argument in so far as the dam with a 1 200-ft. radius is concerned (with the one exception of making no allowance for yielding foundations). It is believed, therefore, that the statement that no reduction in the usual safety requirements should be made on account of a gravity dam being arched in plan on a radius greater than, say, about 800 ft., has been substantiated.

Some further reference to the Mulholland Dam will be made to prove that the contraction of concrete in a dam is real and not theoretical.

The writer saw the dam on March 10, 1926. It was completed in 1925. The maximum height is 200 ft. It is a concrete gravity dam arched in plan on a radius of 542 ft. The elevation of the crest is 750 ft. and that of the spillway, 746 ft. The water stood at Elevation 716 ft. or within 36 ft. of the spillway crest. No contraction joints were placed in the structure.

Numerous cracks extending from the foundations to the crest and from the front to the back face, in so far as the front face could be observed, had opened up and were still open, although the dam was under about four-fifths of its full head. While these cracks did not interfere with the usefulness

of the structure, it was evident that no benefit was being derived from the arch at that time.

This condition might not be true of a higher dam arched on the same radius since the cantilever deflection at the top, say, for a 300-ft. dam, would have been sufficient to have closed the cracks theoretically at least, thereby developing arch action. Arch action in the upper part of the structure would greatly relieve cantiliver stresses in the lower part.

To illustrate the probability of this latter point, some preliminary data in regard to the deflection of the Don Pedro Dam of the Turlock and Modesto Irrigation Districts of California, will be given (see Table 2). This experiment is not as yet complete, but, in so far as it goes, it is believed to be accurate. It was made under the direction of R. V. Meikle, Chief Engineer of the Turlock Irrigation District.

The dam is a concrete gravity dam arched in plan on a 676-ft. radius. The maximum height is 284 ft. The section and plan are very similar to that of the Exchequer Dam, of which it appears to have been the model.

The deflections were measured on July 24, and August 1, 1923 (Table 2). Most of the concrete had been placed prior to October 1, 1922, although a gap had been left at the crest, about 90 ft. high and 32.5 ft. wide, which was closed prior to March, 1923.

TABLE 2.—DEFLECTION MEASUREMENTS, DON PEDRO DAM.

Date.	Elevation of water, in feet.	Temperature.		Deflections.		
		Air.	Water.	Point No. 1, Elevation 1172.	Point No. 2, Elevation 1099.	Point No. 3, Elevation 1049.
July 24, 1923	1 158.45	.....	.....	0.022 ft.	0.029 ft.	0.0075 ft.
August 1, 1923	1 157.95	70° Fahr.	16° cent.	0.019 ft.	0.028 ft.	0.018 ft.

Three plumb-bobs were embedded in the down-stream face with their points projecting vertically at Elevations 1172, 1099, and 1049, respectively (assumed datum). The first elevation, 1 172 ft., is on top of the concrete wall on the lower side of the roadway crossing on top of the dam, the roadway and top of the dam being at an elevation of 1 168 ft.

Three points were located in the rock on the opposite side of the stream from the transit point, one in each vertical plane passing through the transit point and the plumb-bob points. This was done as carefully as possible while the reservoir was empty.

It will be noted from Table 2 that the deflection at the top under approximately full load is less (by 0.009 ft., or 0.108 in.) than the deflection at a point 73 ft. lower in this dam in the second case and almost this in the first case. This is a material difference well above the limits of error of the observation. Since the dam under cantilever stresses alone could not deflect less at the end of the cantilever than at some point below the end, this is conclusive proof that arch action was being developed at that time.

The question might well be raised in this case as to whether or not the dam had cooled off in the interior. It is planned to make further observations with empty and full reservoir, which will be of great value in settling this phase of the question.

In regard to the third question, it is believed that an allowance should be made for uplift; but that, with grouting the foundations and with proper drainage in the dam and foundations, it is hard to understand how this uplift can exceed the equivalent of 25% of the full hydrostatic pressure at the heel, diminishing uniformly to zero at the toe, except in cases where the foundations are unusually poor.

Modern concrete dams of considerable height are now provided with a drainage channel extending under the full length of the dam immediately down stream from the cut-off wall. An inspection gallery extending the length of the dam, usually at a distance of about 30 ft. from the up-stream face and above tail-water elevation, provides a free outlet for water collected from vertical drain pipes. The pipes should be about 12 ft. apart and should extend from 12 to 20 ft. into the foundations. Also, good practice requires vertical drain holes on about 12-ft. centers, extending upward from the inspection gallery to the top of the dam. Since water seeks the path of least resistance, these drains collect a very large percentage of the water getting through on construction joints or reaching the bottom of the dam under the cut-off wall.

They do not get all the water, however, as is shown by the leaks appearing on the down-stream face of the horizontal construction joints. An example of this leakage is shown, Fig. 17, of the down-stream face of Exchequer Dam after first filling. The usual care was exercised in cleaning the construction joints and flushing them with mortar before pouring new concrete. This leakage decreased very materially after the reservoir had been filled only a short time. The writer has seen similar leakage in other high dams.

An allowance for uplift, varying from at least 10% to at most 25%, should be made except for foundations of a very unreliable nature. The higher figure has the writer's recommendation, even in cases of the best foundations.

As stated by Mr. Jakobsen,* it is impossible for full uplift to occur under the entire dam since the concrete is heavier than the water. If sufficient allowance is made in the design to take care of the initial uplift without tension developing at the heel, there will not be, in any horizontal plane in a block of the dam between any two vertical contraction joints, more than 30 or 40% of the area which does not have contact even with careless handling of construction joints. The drainage system should safely reduce this to the equivalent of less than 25% uplift at the heel, diminishing to zero at the toe, except, possibly, in cases of very porous foundations.

On the other hand, if sufficient allowance was not made in the original design to care for the initial uplift, and tension sufficient to open cracks does develop at the heel, the uplift will increase to full hydrostatic pressure over the surface of the crack, and cause the depth of the crack to increase. Hence, it is important to provide for the initial uplift in the design.

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1142.

FIG. 17.—EXCHEQUER DAM WITH FULL RESERVOIR.

In regard to the section that should be considered in making an analysis of the stresses in a gravity dam arched in plan, Mr. Jakobsen has shown* that the method of selecting a section between radial planes will give results much more closely approaching the actual condition than results obtained by means of the parallel-plane method.

The radial-plane method was used by the engineers of the Los Angeles County Flood Control District in the design of the San Gabriel Dam to be constructed at the Fork's site. Very good reasons for its use are given in a paper by S. M. Fisher, Chief Designing Engineer of the District.† The action in this case is very important, since this is the highest gravity dam now planned for early construction, and a very material addition to the concrete yardage was involved in adopting this method.

Taking a section between parallel planes does not account for water pressure on the up-stream face on the sections immediately adjoining the one selected. This pressure will have a component that would have to be partly absorbed or resisted by it. In other words, the down-stream end of a section between parallel planes 1 ft. apart would be subjected to the pressure from a section of the up-stream face greater in width than 1 ft.

The writer believes that considering sections between radial planes is the most correct method that has been advanced. It does not, however, in itself, provide any additional factor of safety. If it shows tension at the heel, good engineering practice requires that a change in the design be made to overcome this fault.

On the other hand, as confidence increases in the mathematical analysis applied to structures due to study, discussion, and experiment, it is the writer's opinion that higher unit stresses can be allowed, for instance, in compression in the concrete at the toe.

Concrete developing a strength in compression in laboratory tests on 6-in. or 8-in. cylinders of 1 500 lb. per sq. in. at the end of 28 days, should allow working stresses in a gravity dam of 700 lb. per sq. in., or its equivalent of 50 tons per sq. ft., since at 6 months the strength will be greatly increased. While as strong a concrete is not likely to be obtained in the dam from the same mix as in the laboratory, this is more than offset by the fact that it seems safe to assume that a large mass of concrete will safely stand a much greater unit stress than a small cylinder.

The controlling specification for gravity dams up to about 400 ft. in height is the uplift assumption and its effect on the tension at the heel and not the strength of the concrete. Mr. Jakobsen shows‡ the maximum stress in the Exchequer Dam at Elevation 437.5 (which is 272.5 ft. below the crest) to be 504 lb. per sq. in., which the writer believes to be a very moderate stress. He shows the objectionable condition in this case to be tension at the heel even with zero uplift, unless, of course, some of the load is taken by the arch before the cantilever deflects sufficiently to allow the tension to develop. That the uplift assumption governs, is true of all gravity designs of about this height.

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1137.
† *Western Construction News*, July 10, 1927.
‡ *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1141.

Any reasonable section that provides for some uplift without tension at the heel will not show compressive stresses of more than 600 lb. per sq. in.

For dams of much greater height, the stress at the toe becomes one of the governing factors. For sections much more than 400 ft. in height it is necessary to place more material in the upper part of the dam than is required to keep the resultant within the middle third in the upper section, in order to steepen the batter of the down-stream face near the bottom. This is due

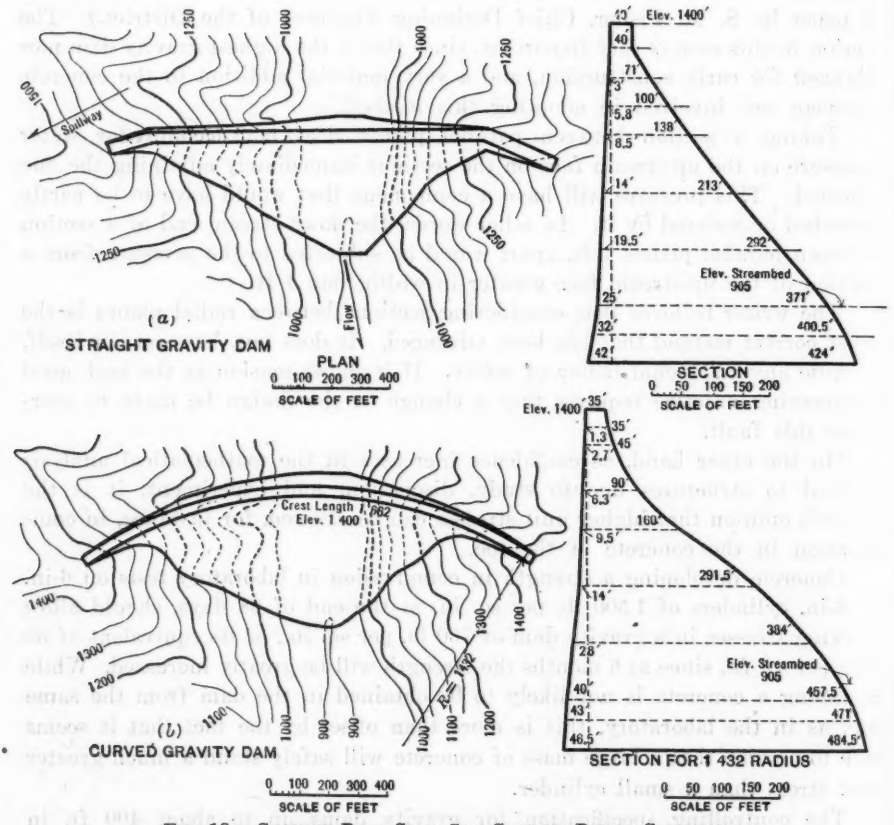

FIG. 18.—GRANITE DIKE SITE, SAN GABRIEL RIVER, CALIFORNIA.

to the Cain theory, now rather generally accepted, which states that the maximum stress at the toe is a function of the angle that the down-stream face makes with the vertical. The well-known formula being:

Maximum stress = maximum stress in horizontal plane $\times$ $\sec^2 \theta$, in which, θ is the angle that the down-stream face makes with the vertical.

The steeper the batter the less the secant will be, and, therefore, the less the maximum stress, other things being equal.

To show the great increase in cost due to curving a high gravity dam, two gravity designs, made by the writer, are submitted (Fig. 18). They were

made for a project requiring a dam 595 ft. high above the foundations and 495 ft. high above the stream bed.

The designs were intended for the purpose of a preliminary consideration of the possibilities of constructing a high dam at the Granite Dike site on the San Gabriel River, California. Exploration of one abutment by tunneling and other factors at a later date caused the elimination of the site from further consideration. These designs are shown on Fig. 18. The estimated stresses are listed in Table 3. The specifications governing the design were not fixed by the writer. They were, as follows: Maximum allowable compression, 40 tons per sq. ft.; uplift assumption, 50% of the hydrostatic pressure at the heel decreasing uniformly to zero at the toe; maximum sliding factor, 0.75; maximum stresses in the down-stream face to be calculated by the Cain formula; and stresses in the curved dam to be calculated by the radial slice method.

TABLE 3.—ESTIMATED STRESSES IN PROPOSED DAM ON SAN GABRIEL RIVER, CALIFORNIA.

Straight Gravity Dam.						Curved Gravity Dam.					
Elevation, in feet.	Depth, in feet.	Pressure at heel, reservoir empty, in tons per square foot.	Pressure at heel, reservoir full, in tons per square foot.	Maximum pressure at toe, reservoir full, in tons per square foot.	Sliding factor.	Elevation, in feet	Depth, in feet.	Pressure at heel, reservoir empty,* in tons per square foot.	Pressure at heel, reservoir full,* in tons per square foot.	Maximum pressure, at toe,* reservoir full, in tons per square foot.	Sliding factor.†
1 400	0	.....	....	*	†	1 400	0	.....	....	.....	....
1 360	40	3.0	1.13	4.25	0.23	1 370	30	2.09	1.05	2.95	0.20
1 305	95	8.26	2.12	8.07	0.46	1 340	60	5.00	1.27	5.83	0.37
1 255	145	11.84	1.55	12.73	0.56	1 260	140	11.49	0.43	13.81	0.61
1 205	195	15.23	2.07	17.58	0.61	1 190	210	15.79	2.79	20.26	0.68
1 105	295	21.68	2.59	24.98	0.65	1 090	310	21.23	6.58	27.27	0.66
1 005	395	28.24	3.69	33.09	0.65	990	410	25.96	7.87	30.60	0.62
905	495	34.88	4.84	40.58	0.65	890	510	31.80	8.34	38.52	0.60
855	545	37.18	3.95	39.70	0 64	856	544	33.92	7.55	34.47	0.60
805	595	38.75	2.27	42.82	0.64	805	595	37.24	5.45	41.10	0.60

*The formula for maximum pressure at the down-stream face, is,

$$F \text{ maximum} = \frac{\text{maximum horizontal pressure}}{\cos^2 \theta}$$

θ = angle between vertical and down-stream face.

†The sliding factor $= \frac{\text{Total horizontal pressure above the joint}}{\text{Total vertical pressure above the joint}}$.

The writer approves of all these specifications except those for maximum stresses and for uplift assumption. In the case of the uplift, it is thought that 25% instead of 50% would have been sufficient. It is believed that 50 tons per sq. ft. would not be too high for maximum stresses.

Fig. 18 (*a*) is a straight dam and Fig. 18 (*b*) is a dam arched with a radius of 1 432 ft. for the up-stream face at the crest. Unless some reliance is placed on the arch, the stresses, uplift provision, and sliding factor are about the same for both structures. The concrete yardage for the straight dam is estimated as 2 720 000 and for the curved dam 3 100 000, an increase of 380 000, or approximately 14 per cent. The yardage estimate is based on the assumption of excavating 50 ft. into the right bank, 20 ft. into the left bank, and 100 ft. below the stream bed in both cases.

This increase in cost is due entirely to arching the dam in plan, provided no departure is made from the specifications. In the writer's opinion, the arching cannot be justified in this case.

The designs shown could, no doubt, be improved and still comply with the specifications. A design showing zero stress at the heel and exactly 40 tons per sq. ft. compression at the toe would reduce the yardage slightly in both cases.

Some reduction could have been made by changing the entire section for about every 50 ft. difference in height since, for heights of more than about 400 ft., material has to be added to the upper part of the section to reduce the stresses at the base. However, this method might have introduced very great arch stresses near the abutments in the curved dam for an increase in temperature much above the closing temperature. It was therefore rejected.

The writer's conclusion is that, what additional safety may be obtained in arching a gravity dam on a radius greater than about 800 ft., is, in most cases, superfluous and not worth the additional cost, except where the foundation is questionable.

All dams curved in plan should be calculated as such and no reduction should be made in the usual safety factor, unless curved on radii so short that it can be shown that arch action will be developed under any reasonable assumption, the burden of the proof being on the designer.

Maximum stresses as high as 700 lb. per sq. in. should be allowed in gravity dams for the usual concrete placed in such structures.

A. J. WILEY,* M. AM. SOC. C. E. (by letter).†—So far as the writer knows, all the arched gravity dams that have been built to date have had their sections increased over a straight gravity section and it has been considered that they were much safer than if they had been given the same section and had been made straight in plan. Mr. Jakobsen's paper is very well timed because at least two important arched gravity dams, recently designed, have had their sections increased beyond that required under the same assumptions for a straight gravity dam.

Of course, it has always been known and admitted that the section of an arched gravity dam between converging radii would not be free from tension in the up-stream face unless there was arch action in the dam transmitting some load horizontally to the abutments. In fact, it has been assumed that there could not be a down-stream deflection of the vertical cantilever without

* Cons. Hydr. Engr., Boise, Idaho.

† Received by the Society, November 16, 1927.

a corresponding deflection of the horizontal arches and a division of the load between the cantilever and the arches.

The author claims* that "as gravity dams are usually constructed, however, the arch cannot act until the cantilever has been broken. * * *" This condition, he claims, is due to the shortening of the arches by setting shrinkage and temperature contractions.

The writer thinks it can be shown that an arched gravity dam is made stronger and not weaker by reason of the arch. To show this, it must be proved that the deflection of the vertical cantilevers with no tension in their up-stream face is more than the deflection of the arch when carrying enough of the water load to avoid tension in the vertical cantilever.

After developing equations for the stresses at the up-stream and the down-stream faces of a section of a dam, between converging radii, in terms of the moment of the water pressure, the weight of the section, and the area and the moment of inertia of a horizontal section, the author applies the equations to a horizontal section at Elevation 437.5 of the Exchequer Dam, where the thickness is stated to be 171.4 ft. In this discussion the writer is using the author's figures, except that he is assuming the height of the arch ring above the base as 52.5 ft. and is using 158 lb. per cu. ft. for concrete because this was the weight of the concrete used in the designs and is also the weight given by the author's Equation (1)† when correctly completed.

It is assumed that the interior of the dam, beyond approximately 20 ft. inside the faces, is practically unaffected either by seasonal temperature changes or by setting shrinkage. Proof of the first assumption is found in the paper by Charles H. Paul, M. Am. Soc. C. E., and the late A. B. Mayhew, Assoc. M. Am. Soc. C. E., entitled "Temperature Changes in Mass Concrete",‡ in which it is shown that beyond about 20 ft., the temperature of the concrete is not sensibly affected by seasonal changes, but gradually approaches the mean annual temperature.

The action of the concrete in the arch due to expansion by the setting heat is somewhat obscure, but it seems reasonable to assume that it is divided between the elastic compression of the concrete and of the abutments and results in up-stream deflection of the arch ring. As the concrete cools, this compression and deflection are relieved, but there seems to be no reason for assuming that it results in tension or in contraction cracks, as would be the case if the concrete could be assumed not to act elastically until it had reached a higher temperature than that of the initial set. In this, reference is made to the concrete in the interior of a large mass and not to the part near the surface where it is affected both by drying and by seasonal temperature changes.

In his discussion of the paper by Fred A. Noetzli, M. Am. Soc. C. E., entitled, "Gravity and Arch Action in Curved Dams", Mr. Jakóbsen states§ that,

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1141.
† *Loc. cit.*, p. 1135.
‡ *Transactions*, Am. Soc. C. E., Vol. LXIX (1915), p. 1225.
§ *Loc. cit.*, Vol. LXXXIV (1921), p. 104.

"* * * in the lower part of high curved dams, the temperature variation is small, especially at the up-stream face, and the effect of lateral deformation due to the weight of the overlying concrete will, in most cases, more than balance any temperature shrinkage likely to take place."

The vertical compression due to cantilever action on the horizontal arch at Elevation 437.5 varies from zero at the heel to 362-lb. per sq. in. at the toe. With a lateral deformation coefficient of one-fifth the resulting compression in the arch is 72 lb. at the toe and zero at the heel, which is equivalent to a temperature change of zero at the heel and 6.5° at the toe. This seems to provide for a much greater range of temperature than exists in the body of the dam.

In the up-stream and down-stream faces of the arch, within a distance of about 20 ft. of the faces, the concrete is affected both by temperature and by setting shrinkage. In the case of two 4 by 5 by 48-in. bars, quoted by Mr. Jakobsen* in his discussion of the paper by Mr. Noetzli previously referred to, one beam, which was exposed to air, shrank 0.0005 in. per in. and swelled 0.00028 in. per in. on being placed in water. The other beam, which set in water, did not shrink at all.

The concrete in the face of a large dam has one face exposed to the air and the other is a part of the interior of the dam. The excess water of the interior forms a source of moisture for the outside part, the rate of setting shrinkage for which should, therefore, be much less than that of small laboratory specimens.

The movements at the contraction joints of the Arrowrock Dam, given in the paper by Messrs. Paul and Mayhew previously referred to, show a combined average setting and temperature contraction of 0.0708 in. in 50 ft., at the face, 0.044 in. at 16 ft., and 0.0324 in. at 22 ft. from the face. This corresponds to a unit shrinkage of 0.00012 at the surface, 0.00007 at 16 ft., and 0.000022 at 22 ft. from the surface; but the last two measurements were taken on exposed surfaces of concrete, subject to local shrinkage and temperature changes in excess of the true shrinkage in the body of the concrete. It should be noted, however, that when these measurements were made, there was still some setting temperature in the dam.

As the unit of expansion of the arch ring, due to lateral deformation from vertical compression at 20 ft. in from the down-stream face, is $\frac{151.4 \times 72}{171.4 \times E}$ $= 0.000032$ ft., which is about 50% in excess of the combined unit contraction found at Arrowrock Dam, it is thought that by neglecting 21.4 ft. of the down-stream part of the arch, the effects of temperature and setting shrinkage will be eliminated and that the intrados of the remaining part of the arch will be under compression from the lateral component of the vertical pressure.

At the up-stream face of the arch ring there is no compression from lateral expansion, but the setting shrinkage and temperature contraction are partly offset by the swelling from wetting.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 103.

The shrinkage at Arrowrock was, at a unit rate of 0.00012, and a unit rate of 0.0002 should be an ample allowance for setting shrinkage at the up-stream face.

For the swelling due to wetting, the same unit rate will be used for the up-stream face so that these two will cancel. The temperature drop in the concrete at the up-stream face may vary from the temperature of the concrete at the time of placing to the minimum temperature of water, or perhaps through a range of 50° at the surface, corresponding to a unit contraction of 0.000275. From the Arrowrock tests the temperature drop, 10 ft. from the surface, would be 8° and at 20 ft. it would be negligible.

Contraction joints at 50-ft. intervals might open 0.165 in. at the surface and 0.026 in. at 10 ft. from the surface, which, it may be noted, is more than twice that observed at Arrowrock. These contractions are confined to the outside 20 ft. of the concrete and do not affect the body of the arch ring.

There is a depth of 272.5 ft. at the arch ring so that the water pressure in the contraction joints, or in any intermediate vertical cracks, would be 118 lb. per sq. in. and the unit pressure in the arch ring would be the same. As this pressure on the arch ring would be confined to the up-stream 20 ft., considered as affected by temperature, it would not cause an up-stream deflection of the arch and can only be considered as effective in converting the up-stream part of the arch (which is affected by temperature), from a state of tension to one of compression while being deflected by water pressure.

Using the author's equations and dimensions, but with a unit weight of 158 lb. for concrete and with no arch action, there would be a tension of 29 lb. per sq. in. in the vertical cantilever at the arch ring.

If it is assumed that the unit weight of water carried by gravity action is reduced to an amount that would give zero tension on the vertical face at the arch ring 272.5 ft. below the surface, this unit weight on the cantilever will be 56.95 lb. per cu. ft. If it is assumed that the remaining weight will be carried by arch action, the unit pressure will be $5.55 \times 272.5 = 1\,512$ lb. per sq. ft.

Using the ordinary formula for cantilever deflection, $y = \frac{p H^3 x^2}{E t^3}$, the deflection at the arch ring is 0.0022 ft.

In his discussion of Mr. Noetzli's paper previously referred to, the author gives a table* showing that the shearing deflections at points below the mid-height in a gravity dam, 200 ft. high, are greater than the cantilever deflections; but the writer finds the shear deflection at the arch ring to be negligible in comparison with the bending, and thinks Mr. Jakobsen may have used E per square inch instead of per square foot, so that his shear deflections may be 144 times too great.

The author has also called attention to the increase in cantilever deflection due to foundation deformation, and to a further increase due to the fact that the plane of the horizontal section assumed in the usual cantilever deflection formula is not normal to the neutral axis and that the resulting deflection is too small.

* *Transactions*, Am. Soc. C. E., Vol. LXXXIV (1921), p. 100, Table 15.

There seems to be no doubt that the cantilever deflection would be increased by both these conditions, but the arch deflection would also be increased by foundation deformation, although probably in a less degree, and consideration has been given only to the cantilever deflection.

Using Cain's equations with shear included, and $t = 150$, $r = 600$, $Z = 14°$, and $E = 288\,000\,000$ lb. per sq. ft., the deflection at the crown of the arch ring is found to be 0.000318 ft. as compared with the cantilever deflection of 0.0022 ft. for the same conditions.

The stresses in the arch ring are as follows: At the extrados, 3.6 lb. per sq. in. compression at the crown and 4.5 lb. per sq. in. tension at the abutments; and at the intrados, 2.0 lb. per sq. in. tension at the crown and 6.7 lb. per sq. in. compression at the abutments.

For equal deflections in cantilever and arch ring the unit weight of water carried by the cantilever would be 37.5 lb. per cu. ft. and by the arch 25 lb. per cu. ft. The deflection of the cantilever from bending alone would be 0.00143 ft. and the arch deflection at the crown with shear included would be the same.

The stresses in the arch ring are as follows: At the extrados, 14.5 lb. per sq. in. compression at the crown and 17.5 lb. per sq. in. tension at the abutments; and, at the intrados, 8.0 lb. per sq. in. tension at the crown and 27.5 lb. per sq. in. compression at the abutments.

The stress on the vertical face of the cantilever at the arch ring with the load divided between the cantilever and the arch to give equal deflections in each would be 14 688 lb. per sq. ft., or 102 lb. per sq. in.

The author claims that open shrinkage cracks at the down-stream face of an arched gravity dam prove that it is not acting as an arch. The writer thinks that this discussion has shown that the shrinkage or contraction cracks in an arched gravity dam extend only about 20 ft. beyond the surface. As further evidence of this it is very seldom that seepage is visible in the open contraction cracks at the down-stream face from full reservoir pressure. Even if they should show seepage, however, this is no evidence that they are not under heavy pressure. There may be uplift under the undrained base of a dam in spite of the fact that it is exerting a greater unit pressure on the foundation than is required in the arches of an arched gravity dam.

It may be claimed that there is temperature shrinkage in the whole body of a dam due to the difference in temperature between that of the concrete at the time of initial set and that of the mean annual temperature of the locality, which the interior temperature of the dam gradually approaches. Considering the quantity of water used in the concrete and its high specific heat compared with that of the other constituents; and considering the constant use of water on the concrete after placing, it does not seem probable that the average temperature at the time of initial set is above the mean annual temperature.

The temperature of the rings, or layers, placed in winter, however, will be considerably less than that of rings placed in summer, while there will be alternate rings placed in the spring and fall that will be of about mean annual

temperature. The tendency, therefore, will be to have constant initial tension in some horizontal zones of the dam and constant initial compression in others, alternating with zones of no initial stress.

These stresses are independent of seasonal changes and after the setting temperatures have been dissipated, they will be constant at all times. The zones of initial compression will tend, when the dam is loaded, to carry more than their proportional share of the arch load; but there will be a tendency to equalize the load by shear between the arch rings.

This unequal initial stress condition might be equalized by pressure grouting, but it would be useless to attempt this until after the setting temperature had been dissipated. As it may require about five years to effect this, the dam will be exposed during this period to these possible concentrated arch stresses. As the arch stresses in an arched gravity dam are very low, the concentrated stresses can be safely carried permanently if desired.

The writer has not analyzed the arch stresses near the top of the dam, but thinks it would improve the design in a gravity type, whether arched or straight, to make the top thicker than customary practice and to compare the cantilever and arch ring deflections and stresses at all heights, using the author's method of computing the cantilever stresses.

W. P. CREAGER,* M. AM. SOC. C. E. (by letter).†—The writer has checked carefully the equations and calculations contained in the author's interesting paper and can find no chance for disagreement except, perhaps, in regard to his conclusions. In order to determine whether or not a gravity dam actually has been made weaker by curving it in plan, it is necessary to investigate the fundamental reasons for providing a factor of safety against failure.

The necessary factor of safety against failure has no fixed value, but, within certain limits, usually is based on the designer's judgment guided by the probable seriousness of the calamity resulting from failure. The cost of a structure increases with the factor of safety adopted. It seems quite evident that a designer would not be warranted in requiring as great an expenditure to insure against unsightly cracks or small leakage as he would to insure against complete failure with resulting enormous financial loss and sacrifice of life.

The author states‡ that "an arched gravity dam will receive some support from the arch and, in most cases, probably sufficient support to prevent actual failure." It is the writer's opinion, based on considerable study, that any properly designed gravity dam having a curvature sufficient to affect its strength as a gravity section materially, would undoubtedly hold as an arch dam in conjunction with its remaining strength as a gravity section, particularly if the joints were properly grouted under pressure.

Calculations by the writer indicate that the Exchequer Dam would probably hold as an arched structure irrespective of its strength as a gravity dam; although even if it were pressure-grouted, unsightly cracks and some leakage might occur before it deflected sufficiently to act as an arch.

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† Received by the Secretary, November 21, 1927.

‡ *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1141.

Is it logical to assume that the Exchequer Dam, even with its slightly reduced strength caused by the curvature, still has an ample factor of safety against cracking and leakage, in consideration of the fact that the element of curvature has provided an increased factor of safety against complete failure? The writer is inclined to believe that it has.

The author states* that "the smaller the radius, the larger is the error". However, the smaller the radius, the greater the ability to act as an arch, and the latter has the greater influence as affecting complete failure.

EDWARD GODFREY,† M. AM. SOC. C. E. (by letter).‡—It is encouraging to note that the author frankly recognizes under pressure as a factor in the design of a dam. His statement that even drainage is sure to be at least a partial failure should give pause to designers who recommend that under pressure be ignored or even to those who would guard against it by installing a system of drainage.

The author deduces the fact that a triangular-section gravity dam will be completely stable against up-stream pressure only if the base is about $\frac{65}{100}$ of the height; and against both up-stream pressure and full under pressure if the base is about $\frac{85}{100}$ of the height. It is significant to apply a triangle on a recently published comparison of the large dams in existence.§ The ratio, $\frac{85}{100}$, will be found to fit all but those that are marked curved in plan. The significant thing is that the straight gravity dams that have withstood water pressure are wide enough at the base for complete stability against both up-stream pressure and full under pressure. The New Croton Dam is included among these, and this dam has several times been publicly held up as an example of a dam the designer of which did not consider under pressure.‖

In that exhibit the dams that are curved in plan do not all come in the $\frac{85}{100}$ class. They are nearer the $\frac{65}{100}$ class. The author maintains that a dam convex toward the water has less stability than a straight dam of the same cross-section. However, in his theoretical demonstration he neglects significant forces which, in fact, should reverse the conclusion.

By analogous processes it can be apparently shown that a dam curved away from the water has greater stability than a straight dam and (as the author has done) that one curved toward the water has less stability. The actual relative stabilities, however, are just the reverse. This is because both these apparent demonstrations neglect forces of vital importance, acting on the element of the dams considered. In the case of the dam that is curved away from the water, there is a circular tension that in any ordinary dam would rupture the concrete and lift it from its abutments. In the dam curved toward the water, the author finds reduced stability. However, he neglects the forces tangent to the curve of the dam. In other words, there is arch

* *Proceedings*, Am. Soc. C. E., Papers and Discussions, p. 1141.

† Structural Engr., Pittsburgh, Pa.

‡ Received by the Secretary, November 22, 1927.

§ *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2356.

‖ Wegmann in *Engineering and Contracting*, December, 1922, p. 126, and *Waterworks*, June, 1923, p. 1302

action which aids very materially in supplying stability. This aid may, in fact, mean the difference between stability and failure.

In the writer's judgment it is the curved plan of some dams, which do not show full stability on the basis of straight gravity dams, that is responsible for their continued existence as dams. Not that this is a proper method of design, but it is the accident of design that has saved some structures. Reliance on it has wrecked more than one.

It cannot be denied that a trapezoidal element of a curved dam must exert pressure on the adjacent trapezoidal elements. The several elements are not free to rotate, and an arc of the dam tends to rotate on the ends of that arc, thus with an increased lever arm. This, however, means increased and unusual pressure against the soil at the ends of the arc which is one of the reasons to be urged against reliance on a mere curved plan.

Any dam must diminish in height toward the ends. If it is founded on rock near the ends, this rock may be utilized as abutment, and then the curved plan (convex toward the water) has undoubted and very real advantages. Arch action can be utilized under such conditions.

However, if the foundation is not rock, attempts to make long thin arch dams may result in disaster. Two dams of this type recently failed.* One of these two similar dams was 236 ft. long and 62 ft. high at the highest point. The whole dam weighed about 6 000 000 lb. while the thrust of the arch at the mid-section was 18 000 000 lb. Much of this would travel to the ends and to the small abutments. The estimated pressure on this dam in the direction of the axis of the stream was 9 000 000 lb. It is not to be expected that a dam of these proportions would stand; but if it had been built of a cross-section approaching that required for a gravity dam, the curved section might have been the salvation of it.

Real stability in a gravity dam is attained when the width of the base is made about $\frac{85}{100}$ of the height. Failure and freezing of drainage systems can then be forgotten. No dam of these proportions has ever failed.

C. A. P. TURNER,† M. AM. SOC. C. E. (by letter).‡—Published measurements on the experimental arch dam at Stevenson's Creek, California, having a vertical up-stream face, should furnish an opportunity for testing Mr. Jakobsen's formulas and the accuracy of the assumptions involved.

The tensile fiber stress on the up-stream face when the dam was full, may be roughly computed according to his formulas at from 6 000 to 10 000 lb. per sq. in. Yet the concrete held together and the dam carried the pressure without injury and was later over-topped in a flood. The computed down-stream deflection under this stress in the central part of the dam is from fifteen to fifty times greater than that measured when the dam was full. The quarter-point presented no down-stream deflection whatever, while between the quarter-point and the end of the dam the crest deflection was up stream against pressure, instead of down stream as computed by Mr. Jakobsen's notes.

* *Engineering News-Record*, October 14, 1926, p. 616.

† Cons. Engr., Minneapolis, Minn.

‡ Received by the Secretary, November 30, 1927.

This divergence of computed and measured results indicates that the predominant resistance of an arched dam, having the relative height and length of the test dam, is wholly different from that of the straight one, as assumed in the derivation of the formulas under discussion.

Consider, for example, a dam, straight in outline, that is, two or three times as long as it is high, placed in a rectangular opening in rock. Under the water pressure, the center of the dam will deflect down stream. The base, being held rigidly by the rock bottom, will not move and the ends of the dam, likewise being held rigidly by the rock sides, do not deflect. Consequently, the dam is both bent and warped or twisted. In such warping and twisting horizontal shears in vertical planes are developed, transferring a portion of the applied pressure toward the ends instead of vertically downward to the base.

In an arched dam, down-stream deflection of the crown or center of the arch cannot take place without twisting the structure. Such twisting shears transfer the reaction of the horizontal forces away from the center toward the ends of the dam and the reaction of the vertical forces shifts in like manner. The down-stream deflection of the top at the center of the dam would tend to twist the crest between the quarter-point and the end of the dam in the opposite direction. Torsion becomes the predominant resistance as opposed to the cantilever gravity action of a straight dam and the middle-third rule cannot be applied rationally to such a structure. Moreover, the curvature of the arch tends to reduce the over-turning moment on each individual element, because the axis of the moment of pressure on the individual elements makes an angle with its neighbor and with adjacent elements so that interference of moments is developed, which accentuate twist and reduce the effectiveness of the applied moment. A component of the pressure on the up-stream face produces a small amount of axial compression, or arch thrust.

If the opening is a V-formation the shift of the reaction laterally places it at a higher elevation than the base at the center or deepest part, thereby reducing the lever arm of the over-turning forces. Thus, an arched dam presents elements of resistance that the straight dam lacks. It permits an economy of material that cannot be secured when constructing a straight dam.

Rankine, in applied mechanics, states the premise on which the commonly accepted theory of flexure is founded; to wit, that the beam be subjected to transverse forces only and that the length be fifteen or more times greater than the depth. Then, shear strain may be disregarded as negligible from the practical standpoint only in determining the elastic deflection and maximum tensile and compressive fiber stresses.

In a triangular gravity section, such as that considered by Mr. Jakobsen, the ratio of base to height of strip exceeds the limits under which shear strain may be ignored. Again, because the weight of the strip constitutes a longitudinal force greater than the transverse force, the condition of transverse force normal to the axis only, under which Rankine demonstrates the ordinary theory to be approximate, is lacking.

The assumption on which the ordinary theory of flexure is based, differs widely from the elastic theory. The shearing modulus of the material, F, is

assumed to be infinite, an imaginary value, in place of its true value of approximately three-eighths of the modulus of tension or compression. The utility of the ordinary theory is confined to the determination of the maximum fiber bending stress, at a section where rhombic shear strain passes through zero. It is not adapted to the determination of the elastic behavior or elastic stress in a beam subjected to longitudinal thrust.

For example, a wood strut, supporting concrete work, is more rigid laterally when loaded reasonably than when it is carrying little or no weight. If its deflection is to be computed correctly the moment of inertia to be applied differs from that permissible under the condition of transverse force only without axial strain, and the idea* that the center of gravity of the section constitutes the axis of internal rotational resistance must be discarded. Every attempt to solve the problem of elastic resistance by assuming perfect elasticity of material under tension and compression and no elasticity at all under shear stress invariably leads to inconsistency and error.

Arch action in an arched dam is computed by some on the assumption that no shear may be transmitted by the concrete longitudinally along horizontal planes; but if the concrete can transmit no shear longitudinally (since the material has the same resistance to shear transversely in the same plane), cantilever resistance would be an anomaly unrelieved by an additional assumption that the concrete cannot resist horizontal shear in vertical planes on which, in turn, twisting resistance depends. Thus, all assumptions, differing unreasonably from a complete acceptance of the natural characteristics of the material dealt with, lead to theoretical uncertainty and confusion.

To determine the predominant resistance of an arched dam it is necessary to consider the fact that the base of the dam on a rock formation is fixed. It does not slide down stream and it cannot be deformed† lengthwise along the rock bottom. This eliminates the idea of the predominant action as an arch, provided the height is a fraction of the length. Any strip above the base which is shortened by down-stream deflection would involve, of necessity, longitudinal horizontal shears in horizontal planes represented by twisting couples about horizontal axis normal to the face of the dam. The shift of the reaction along the arch results in twisting couples on a normal section about the longitudinal axis of the structure and, in turn, horizontal shears in vertical planes involve a twisting couple about a vertical axis. A three-dimensional twisting strain is the result.

Computing torsional resistance involves the polar moment of inertia of the cross-section. In the case of the Stevenson Creek Dam, the polar moment of inertia resisting torsion is enormously greater than the transverse moment of inertia resisting cantilever action. Torsion accounts for the precise kind and character of deflections measured, which may not be accounted for by either arch action or cantilever strip action.

As previously noted, the stress approximately computed on the cantilever principle for the test dam would be thirty to fifty times as great as the ulti-

* This erroneous idea apparently constitutes a premise in the derivation of Mr. Jakobsen's formulas.

† The section of the rock base being far greater than the dam, its elastic compression becomes relatively negligible.

mate resistance of the material; and yet it does not appear from the reports to have been cracked where the greatest tensions from torsional stress would naturally occur; that is, nearest the axis of rotation. It is to be borne in mind that the shearing resistance of concrete is far greater than the tensile resistance and one does not have to assume an imaginary tensile resistance to accompany an imaginary infinite value of the shearing modulus in order to make the computed and experimental results reasonably consistent. The fact that the experimental dam was relatively thinner than many existing arched dams would in no wise alter the character of resistance involved, because the increase in polar moment of inertia increases with the transverse moment of inertia.

Relative thinness accentuates the deflections and permits a rational deduction therefrom, which would be more difficult to make were the experimental dam thicker and the deflections too small for accurate measurement and relative comparison. In considering the torsional resistance of an arched dam structure, the engineer may well investigate the form of cross-section which will produce the minimum stress in the material. He will avoid the expensive warped surfaces embodied in certain types of design and revert to a cross-section more readily built and more nearly approaching the rectangular or trapezoidal section.

The reverse curvature of the crest deflection of the experimental dam has been attributed by some to a buckling tendency due to axial compression, but it is impossible to harmonize such a tendency with the condition of uniform hydrostatic pressure normal to the arch. Again, the exact theory of elasticity negatives such an explanation because where shear strain exists, as it must in a section of this kind fixed at the end, there is a definite limit to the number of changes of curvature and points of inflection which approximate theory fails to indicate and which are exceeded in the reported results.

A thin steel spring under compression fixed at its ends will buckle with two points of contraflexure only. To deflect, as the crown of the experimental dam was observed to deflect, would require four points of inflection because the end of the arch where the height is small was fixed in position.

Toward the conclusion of his paper, uncertainty regarding the distribution of shear at the base of the dam is noted by Mr. Jakobsen and he explains that he has purposely excluded this from his discussion, desiring only to show the influence of curvature when his assumptions were maintained. This is unfortunate because the magnitude of the resisting moment and the overturning moment depends on the relative location, position, and elevation of the reaction which determines the lever arm of the external forces.

The principle that an applied force is transferred to its support in a linear direction by the internal stresses of shear, twisting, and bending, and by the shortest path that the cross-section and rigidity of the material permit, may perhaps be utilized in mathematical form to locate the distribution of the reaction with reasonable precision, as has already been accomplished in the exact theory of plate action.

The Stevenson Creek model dam was undoubtedly selected by the engineers designing it as an average type of structure. The relative proportion

of length to height and the slope of the base at the sides determine predominant action. With an angle of less than 45° to the vertical, with a height from the base to the crown two or more times the chord, the arch would modify the resistance such that the twisting of the plate would transfer a major portion of the applied force to the sides rather than to the bottom. Then, the relative moment of twisting resistance, as compared to arch resistance, would decrease so greatly for median or upper layers, that the dam with some propriety might be treated as if the predominant resistance were that of arch strips.

As engineers become more convinced of the economy of combining metal and concrete, making the concrete able to resist tension and withstand temperature stress without disintegration, and the ease with which the base of a reinforced concrete dam may be anchored into a rock foundation, construction of gravity dams will become less frequent and the monolithic reinforced structure will replace the concrete gravity dam.

The objections commonly raised to such a design may be summarized, as follows:

First.—That the steel is likely to rust and the expansion of the rust will crack the concrete and cause it to disintegrate even if the embedment is relatively deep.

Second.—That the concrete may be disintegrated by frost and, therefore, plain mass concrete is less objectionable from this standpoint. The writer's observation has been that while reinforced concrete disintegrates by the expansion of scale rust on bars that have been exposed and have not been properly cleaned before embedment, there are no well authenticated examples of such disintegration where proper care and precautions have been taken. Along the coast, where the salt air tends to increase the formation of rust, the labor involved in properly cleaning the steel is much greater than the cost of giving it a protective coating before shipment from the mill such that no rust may be formed upon it during the time it is exposed to the weather before embedment. As such coating of the proper kind increases rather than decreases the bond between the concrete and metal, it may be regarded as advantageous and economical.

A dam 3 000 ft. long, well reinforced, without expansion joints, subjected to a range of temperature from 50° below zero to 100° Fahr., above, has remained tight after a number of years, so that the objection to monolithic action on the ground that temperature changes would prevent such action, seems to be without foundation.

The engineer desires a precise mechanical theory for the computation of the elastic behavior and for the resistance of the structure he designs whether it be a frame or a concrete dam; but the computed elastic behavior depends on the values assigned to the elastic coefficients of the material.

The assumption that a plane normal cross-section of the unit strips treated by Mr. Jakobsen remains plane after flexure without rotation from the normal requires an infinite value of the shear modulus, F.

Because deflection of the crest at the center of the dam is greater than that at the quarter-point, or toward the end, shear distortion is involved and the value of the modulus of shear which would permit distortion in the assumed absence of stress on the vertical normal plane between unit strips is zero. Hence, Mr. Jakobsen's premise consists in treating the shearing modulus, F, as varying in the same identical concrete from zero in a vertical plane to infinity in a horizontal plane. The inconsistency thus involved renders the theory without foundation or value as an elastic analysis.

While arch action and bending are affected by cracking of the concrete, torsional resistance may be transmitted by keying adjacent sections together as is customarily done by those who are prejudiced in favor of expansion joints.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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SOME PHASES OF IRRIGATION FINANCING

Discussion*

By D. C. Henny, M. Am. Soc. C. E.†

D. C. Henny,‡ M. Am. Soc. C. E. (by letter).§—The writer is under great obligation to the many distinguished authorities on the subject of irrigation for their discussions. He is gratified to find that the presentation of a paper, rather economic than technical in its nature, is commended; that it is not, for that reason, criticized; and that, on the whole, there appears to be fair agreement on the conclusions reached.

The reasons stated in the discussion for approval and resulting from greatly diversified experience and from viewing the subject from various angles, add great weight to the conclusions.

Mr. Hinderlider‖ raises the question as to what is profitable farming. The writer believes that ability of the average industrious and intelligent farmer to raise a family according to reasonable modern standards and to keep out of debt, private or public, may be regarded as complete success even without accumulation of money. It is in this sense that the term, profitable farming, was used in the writer's Conclusion, (*a*)¶ that "increase of irrigated area will and should be slow when and where crop prices do not render farming profitable". The Government, whether State or Federal, should not encourage, by subsidy, or otherwise, any increase of opportunities which would force a lowering of the standard of living for the average man.

Mr. Hinderlider questions** the writer's conclusion that "the irrigation district method is not well adapted to the reclamation of large areas of desert

* Discussion on the paper by D. C. Henny, M. Am. Soc. C. E., continued from November, 1927, *Proceedings*.

† Author's closure.

‡ Cons. Engr., Portland, Ore.

§ Received by the Secretary, November 25, 1927.

‖ *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1351.

¶ *Loc. cit.*, May, 1927, Papers and Discussions, p. 912.

** *Loc. cit.*, August, 1927, Papers and Discussions, p. 1352.

land". In support of his criticism, he quotes developments in Colorado. He is far more familiar with Colorado conditions than the writer who cannot directly question the statement advanced that "many of the largest and most successful developments in Colorado have been effected under the original Irrigation District Act and amendments thereto".

The writer has some familiarity with conditions as they existed in Colorado thirty and forty years ago. At that time irrigation, after making great strides in the vicinity of Denver and Pueblo and on the Upper Rio Grande, all under private financing, received a severe setback through financial losses, partly resulting from the panic of 1893. Mr. Burkholder happens to refer to this subject where he states* that, "many of the private companies effecting irrigation work in the South Platte Valley, near Denver, Colo., became bankrupt in the early stages of the development". The foundation thus laid was subsequently built upon by District organizations, which is quite another matter than starting reclamation from the virgin desert stage by District financing. Mr. Hinderlider does not state definitely to what extent the latter method by itself has proved successful in Colorado. The point which the writer tried to make is not refuted by the success which may at times have been achieved. The risk of failure is present in most cases to a high degree, as was well shown by the statistics compiled by the late R. P. Teele, Esq., and published and referred to in his discussion.† It was also fully confirmed by other discussors of the writer's paper and also by the contemporaneous paper‡ of R. K. Tiffany, M. Am. Soc. C. E., and the discussions thereof.

The point is that the security at the time of bonding is practically all potential and that District bonds cannot be sold at reasonable figures unless the prospective buyer is kept in ignorance of the risk. The apparent necessity for concealment as well as the large percentage of failures which have resulted in such cases appear to the writer to furnish ample justification for his conclusion.

Mr. Hinderlider's argument§ in favor of turning over, at the earliest practicable moment, the operation and management of subsidized irrigation enterprises to District organizations is fully concurred in and has for years been the process of the U. S. Reclamation Bureau in spite of frequent local opposition.

Mr. Grunsky§ argues that a Government subsidy consisting of any reduction from the full cost of irrigation, together with interest, is promptly absorbed by the land owner in the price he obtains for his land when he sells. This has been true for many years, but does not hold true for the last five years because of inability to sell at acceptable prices. It may also be stated that earnest endeavors are being made to counteract such absorption. One plan is to require of original land owners, before project work is commenced, an agreement that in case of sale the true selling price must be stated in the

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 2051.

† *Loc. cit.*, p. 2047.

‡ "State Reclamation in Washington," *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 913.

§ *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1352.

deed, in order to make a transfer legal and that at the time of sale, part payment must be made of water charges otherwise not yet due, in case the selling price exceeds a given base.

Other plans intended to hinder land speculation have been introduced by District organizations. The desirability of giving the ultimate farmer the benefit of the subsidy is fully appreciated and will probably in some manner and degree be realized in the future.

The writer is especially gratified in finding full support from Mr. Grunsky, Mr. Newell, and others of his contention that in case of subsidy the total extent of the commitment should, so far as practicable, be determined and made known. The difficulty of accomplishing this was forcibly pointed out by Mr. Newell in his statement* referring to past policies as to Government officers, that "discussion of real costs may be followed by dismissal". The writer also is under popular disapproval in certain sections in the West for his insistence on this important requirement without which all subsidy becomes illegitimate and baneful.

Mr. Jacobs† would prefer subsidy to be confined to the loan of funds for a long period and at a low rate of interest, 3½ or 4 per cent. He believes that "most of the projects that can be justified at all, can work themselves out to economic safety without the aid of any subsidy". Where this is true any subsidy, whether consisting of a low rate of interest, or no interest at all, is undesirable provided the time element is not seriously affected. The truth of Mr. Jacob's contention is, however, difficult to ascertain. It is certain that future expansion of irrigation will be greatly retarded if interest payments are insisted on and economic feasibility is determined by disinterested and well qualified men. It is an open question, however, to what particular extent subsidy should be granted at any particular time and place, and it is admitted that any type of subsidy is liable to serious abuse.

Mr. Dunham‡ comments on publicity which unavoidably advertises contemplated Government activities. The effect of resultant land speculation is, of course, harmful and has been combatted so far as possible by withdrawal of public lands and by exacting contracts from present owners. Quiet accomplishment under methods flavoring of Secret Service is, however, believed to be impracticable as well as undesirable.

Mr. Newell§ reviews the history of the Reclamation Bureau from an angle which, even where it goes far beyond the original scope of the paper, makes his discussion a most valuable addition. His extensive experience leads him to approve the writer's conclusion that it is important to sever politics from business in the administration of a Government subsidy.

Mr. Stevens points out‖ that if Government subsidies consist of the loan of money free of interest "such subsidies are more than repaid by the general increase in taxable values resulting from conversion of raw to productive

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1746.
† *Loc. cit.*, August, 1927, Papers and Discussions, pp. 1355–1356.
‡ *Loc. cit.*, p. 1356.
§ *Loc. cit.*, September, 1927, Papers and Discussions, p. 1745.
‖ *Loc. cit.*, p. 1751.

lands". This is an incidental benefit of unquestioned value, but cannot in itself be an argument in favor of subsidy if it is unwise in other respects.

Mr. Stevens holds that the writer's summary* should have included the necessity for studying all elements of cost in advance. Conclusion (*h*) ends with the words "total cost, including all items of present and future expenditure". This expression is believed to be sufficiently comprehensive.

He also suggests a change in Conclusion (*d*) to the effect that the ill adaptation of the irrigation district method to the reclamation of large areas of desert land should refer to large areas in single projects. This was intended to be the meaning and was so understood by others discussing this point.

Mr. Teele† stated that irrigation in the West is not solely for the purpose of providing for local food production, and that a far smaller area than is at present irrigated could satisfy this requirement. This is quite true, but whether exported fruit or local food be considered the by-product, it is unquestionable that either is of benefit to the other and will permit profit at lower selling prices than if production is confined, in spite of suitability of land, to one or the other alone. The writer knows of no extended irrigated area devoted solely to production for export.

Mr. Teele in his interesting discussion further argued that 31 000 000 acres, which went out of cultivation during the period of 1920 to 1925, will act as a "shock absorber" when need for more agricultural products should become evident. This is believed to be the case to a far less extent than the acreage figure would indicate, as the greater part of this area is probably land, marginal because of being situated in the semi-arid belt and, therefore, without irrigation being capable of producing a limited character of crops, mostly cereals. When shortage of food should first become seriously felt in the United States it will probably not be because of shortage of such products. It seems rather probable that, coincident with the gradual re-use of this area and in part preceding it, additional irrigation will be necessary to preserve a desirable balance.

Mr. Teele found the general Western demand for new projects was explained by the easing off in the enforcement of financial obligations accepted by the farmers. This may be one reason, but it is by no means the predominant one. The demand generally results from the desire to realize on local assets supposed to be valuable, but now lying idle. It is probably made partly with the knowledge that past policies in this respect are changing for the better and mostly in ignorance of the exact showing made in regard to collections.

Mr. Teele further explained what he meant by local subsidy in his *Bulletin No. 1257*, U. S. Department of Agriculture, 1924, to the effect that this term did not refer to local government subsidy, but to a different method of taxation, including not only the land directly affected, but also interests indirectly benefited. In his forthcoming book, "The Economics of Land Reclamation", it is recommended that cost be spread in proportion to benefits.

* *Proceedings*, Am. Soc. C. E., May, 1927, Papers and Discussions, p. 912.
† *Loc. cit.*, October, 1927, Papers and Discussions, p. 2047.

It is questionable whether the term "subsidy" is correct in such a case. A taxpayer does not ordinarily consider himself as a contributor to subsidy when he pays his improvement tax. However, the method of taxation on the basis of benefit has decided advantages whether in the case of irrigation alone or in connection with flood control and drainage, as was ably argued by Mr. Burkholder.

This method would make possible reclamation under the District bonding method where otherwise it would be of questionable legitimacy. It contains the elements of inherent justice. To the extent that its use is practicable and a fair division of benefits can be worked out agreeable to all interests it will reduce the need of governmental aid. Mr. Burkholder correctly points out, that this aid is inadvisable where it is not necessary. This method is likely to be on trial, as explained by Mr. Burkholder, on the Middle Rio Grande, and there is good reason for believing that it will work out successfully and that in many cases in this manner Class "B" Districts may be raised to the "A" class.

Mr. O'Brien* enters into the wider field of the Malthusian theory. He argues that development in any section ahead of actual need of the whole country is premature and should not be countenanced, even if admitting that development of the West will strengthen America's position in the Pacific Ocean. If this theory had been applied during the earlier Colonial days, the United States would probably not extend beyond the Alleghanies. There would have been no free land grants on the part of the United States and no general encouragement of Western expansion. Subsequently, there would also have been no need of railroad land grants, food products having been relatively abundant ever since the first successful settlements west of the Atlantic. He believes that the United States should hold a large portion of the West as a reserve for future use. The writer holds that, ethically, the Nation cannot maintain a claim to any large portion of this continent against claims of other nations who may need it now, if held merely for future use. Many, especially in the West, may be misled by the urge of further early development, but they believe it to be founded on sound principles which dictate that the only right they have to the land they claim comes from occupation and beneficial use.

Mr. O'Brien considers that high living standards may become a peril in the future and, by implication, that they should be held lower to train Americans for effective competition in the future when population will press against the limits of food supply. Others, however, live in hope that science and experience may yet teach people how to ward off the fatal pressure against food limits otherwise logically predicted and to maintain and even raise living standards for all.

Mr. O'Brien advocates guaranty by States benefiting from irrigation development. If this benefit radiates from its center like waves in a pond, who shall say that it extends merely to an artificial delimitation of the pond and not to the shore line. Independent of this view it should be repeated that, comparatively speaking, Western States, in the money sense, are poor

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2337.

and that in the aggregate they contain an enormous area of Government land which receives benefits from all improvements, but pays no taxes. State aid, where extended, will probably have to be confined to help of worthy Districts in financial difficulties, origination of small new projects and assistance in colonizing large Federal projects.

In this matter of subsidy there is moreover apt to be an adjustment. The argument used by Mr. O'Brien would be even more effective if applied to the case of Government expenditures for the Harbor of New York, which surely benefits that wealthy city and State to a far greater extent than it does the West; also, to the case of tariffs which protect and benefit one industry more than another and all industries more than farming. The West is rather an inopportune section of the United States upon which to visit a strict limitation of Government aid, not applied elsewhere.

In various discussions, attention is called to the different nature of subsidy for the reclamation of private lands and subsidies, such as railroad grants, tariffs, and harbor improvement. This same point may be raised in connection with Mississippi River flood control. No one subsidy can be exactly like another, and the only question involved before it is granted is whether it is, on the whole, for the good of the Nation.

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THE LAKE WASHINGTON SHIP CANAL, WASHINGTON

Discussion*

BY L. C. SABIN, M. AM. SOC. C. E.

L. C. SABIN,† M. AM. SOC. C. E. (by letter).‡—This comprehensive and interesting paper has one serious defect. The entire installation worked so well that nothing is left for adverse criticism. Possible exceptions to this general statement are the complications arising from the fact that the upper pool is fresh, while the lower pool is sea water, and the suggestions of the authors that the laterals from the main culverts should have been distributed in a different manner.

The former condition has been carefully studied and practically corrected. The latter seems not to have received corresponding attention, and it is to this feature that this discussion is addressed. In lock design there are several considerations that render it impracticable to predetermine accurately by theory the action of a certain proposed arrangement of culverts, valves, and ports or laterals. For the emptying culverts, where the flow is from a restricted chamber to the open canal, the arrangement seems to be comparatively unimportant as to its effect on surge in the chamber; but in filling, considerable and variable differences in level may occur at the two ends of the lock with consequent objectionable currents within the chamber.

According to the data given in the paper there are two culverts in the 900-ft. lock, each having an area of 108.5 sq. ft. The ports or laterals are generally 4 ft. wide and 2 ft. high but eight of the laterals are 6 ft. high to afford easy access to the culverts for inspection and repairs to valves. Four of the larger ports and twenty of the smaller ones enter the upper chamber, 450 ft. long, while four of the larger and twenty-four of the smaller ports enter the lower chamber, 375 ft. in length.

* Discussion on the paper by W. J. Barden and A. W. Sargent, Members, Am. Soc. C. E., continued from December, 1927, *Proceedings*.

† Vice-Pres., Lake Carriers' Assoc., Cleveland, Ohio.

‡ Received by the Secretary, November 3, 1927.

Considering the lock as a whole the ratio of port area to horizontal cross-section of the chamber is approximately:

$$\frac{(2 \times 6 \times 4) + (4 \times 2 \times 20)}{450 \times 80} = 0.0058$$

for the upper portion, and

$$\frac{(2 \times 6 \times 4) + (4 \times 2 \times 24)}{375 \times 80} = 0.0080$$

for the lower portion. The authors state* that,

"The idea was that this would result in a more nearly uniform filling of the lock. In practice, it appears that the filling takes place faster in the lower end, resulting in a slight up-stream current in the lock. This has not been particularly troublesome, but in a new design the number of laterals in the lower half should probably be made the same as, or even less than, in the upper half."

In the Pedro Miguel Lock of the Panama Canal the main culverts discharge into small laterals under the floor which, in turn, communicate with the chamber through ports through the floor. Disregarding the area between service and guard-gates and considering the chamber as a whole, there are in the upper half, six laterals from the side culverts and four laterals from the center culvert, each having an area of 41 sq. ft. (although apparently restricted by openings from the culvert having an area of but 33.2 sq. ft.), giving a total area of laterals of 410 sq. ft. In each lateral there are five ports having an area of 12.24 sq. ft. each, or a total area of ports of 612 sq. ft.

In the lower half there are five laterals from the side culvert and six from the center culvert, giving a total of 451 and 673 sq. ft. for laterals and ports, respectively. The ratio of lateral area to horizontal cross-section of lock is thus, approximately:

$$\frac{410}{110 \times 500} = 0.0074$$

for the upper half, and,

$$\frac{451}{110 \times 500} = 0.0082$$

for the lower half. The ratio of port area to horizontal cross-section of lock chamber is approximately 0.0111 for the upper half and 0.0122 for the lower half. There is nearly equal distribution of ports throughout the lock, but with more ample provision of port areas than in the Lake Washington Lock.

In the latest locks at St. Marys Falls Canal there are six culverts under the floor in each lock. The outer culverts have forty ports each and extend only about half way down the lock. The other four culverts extend the entire length of the lock and each is provided with fifty-nine ports spaced 30 ft. apart at the upper end, this spacing being gradually reduced to 15-ft. centers at the lower end. Each culvert has a net cross-sectional area of 88.07 sq. ft. and the ports are each 1 by 3 ft. In the upper half of the 1 350-ft. lock chamber there are about 178 ports fed by six culverts and in the lower half about 138 ports fed mainly by the four central culverts. The design con-

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1248.

templated that in actual operation the valves of the central culverts would be opened first and those in the outside culverts only when filling had progressed so far that ample opening under the reduced head would not result in surge.

The fact that the several culverts communicate with each other near the upper end to relieve side pressure on the culvert walls complicates any analysis of flow conditions when only part of the valves are opened, but when all the valves are open, the ratio of port area to horizontal cross-section of lock chamber is:

$$\frac{178 \times 3}{80 \times 675} = 0.0100$$

for the upper half, and,

$$\frac{138 \times 3}{80 \times 675} = 0.0077$$

for the lower half.

For these three locks the ratios of port area to horizontal area of lock chamber are given in Table 5.

TABLE 5.—Ratio of Port Area to Horizontal Area.

Lock.	Upper half of chamber.	Lower half of chamber.
Lake Washington	0 .0058	0 .0080
Pedro Miguel	0 .0111	0 .0122
Sault Ste. Marie	0 .0100	0 .0077

In the Lake Washington Lock, with the area of ports in the lower half of the chamber 138% of that in upper half, the authors state:* "It appears that the filling takes place faster in the lower end, resulting in a slight up-stream current in the lock."

In his paper on the action of the water in the Panama locks† H. F. Hodges, Maj.-Gen., U. S. A. (*Retired*), M. Am. Soc. C. E., states:

"When any one of the locks is filled, using the side wall culvert only, the discharge is noticeably greatest at the opening in the floor farthest from the culvert, and nearest the middle wall. The velocity of flow in the laterals evidently carries the water past the first two or three openings without giving them their full share of the discharge. Were the laterals lengthened, doubtless this effect would become less and less marked; and, finally, the maximum discharge would be found at some opening nearer the source of supply. With the proportions which they have, however, there is no doubt that the openings nearest the middle wall discharge a larger volume of water than those nearer the side wall, when the side culvert only is used. The result is a slight slope of the water's surface, which draws vessels away from the middle wall toward the side wall. This effect is not serious and disappears when both culverts are used.

"The side culvert occupies much the same relation to the laterals which draw from it as does each lateral to the openings through which it discharges.

* *Proceedings*, Am. Soc. C. E., August, 1927, Papers and Discussions, p. 1248.

† "Action of Water in Locks of the Panama Canal," *Professional Memoirs*, Corps of Engrs., U. S. A., Vol. VII, No. 31, p. 18.

The side culvert is, however, made longer, and it is not plain to the eye which one of the laterals gets the largest share of the water at any instant. The ones farthest up stream discharge first, the boils from these beginning to show in about one minute after the first motion of the valves. The flow becomes apparent to the eye progressively down through the length of the lock, the lateral at the lower end—that is, the one farthest from the filling valve—showing flow last of all."

At the Sault locks,* where the area of ports in the lower half is only 77% of that in the upper half, it appears that when filling with either four or six valves the boils begin to show first at the upper end and the water at this end momentarily rises faster, but the water in the chamber is nearly level for the first minute, the down-stream end is then higher for about a minute, the maximum difference being nearly 2 ft. Thereafter, the level fluctuates back and forth, the water being first higher at one end and then at the other, with a maximum difference of about 1 ft. and gradually reducing.

In view of these several observations it appears that immediately after opening the valves the ports at the upper end of the locks will begin to discharge first, but as soon as the inertia of the water has been overcome the level will be higher near the down-stream ends of the culverts for a short time. This seems to hold true whether the ratio of port area to chamber area at the lower end is 40% greater or 23% less than the same ratio for the upper end. Accordingly, it appears to be impracticable to correct the situation by variation in this ratio in the design of the culverts.

Unless this fluctuation is extreme the effect ordinarily is not great, but it has been entirely overcome at the Sault by the method of operation. At first, the plan adopted was to open the central pair of valves first, the intermediate pair after 3 min., and the outside pair 3 min. later. This confined the rate of rise in the chamber to a maximum of 4 ft. per min. Later, it was found that still smoother operation could be secured by opening each pair of valves very slowly, consuming about 1½ or 2 min. in opening each pair. By this method the tendency to surge is eliminated and only very moderate stress on the mooring lines is required. While the lock can be filled in about 6 min. by opening all valves simultaneously, about 10 min. are consumed in ordinary operation.

TABLE 6.—Comparison of Culvert and Port Designs.

Subject.	Lake Washington Canal.	St. Marys Falls Canal.	Pedro Miguel Lock.
Horizontal dimensions, in feet	80 by 825	80 by 1 350	110 by 1 000
Horizontal area, in square feet	66 000	108 000	110 000
Net Area:			
Main culverts, in square feet	217.0	528.4	510.0
Area ports, in square feet	448	948	1 285
Ratio, culverts to chamber	0.0033	0.0049	0.0046
Ratio, ports to culverts	2.06	1.79	2.52
Ratio, ports to chamber	0.0068	0.0088	0.0117
Approximate time, in minutes, to fill from 18-ft. head to zero	7⅛	6	5

* "Filling and Emptying the Third Lock at St. Marys Falls Canal," *Professional Memoirs*, Corps of Engrs., U. S. A., Vol. IX, No. 44.

Although the data at hand are not entirely sufficient, a comparison of the culvert and port design and the approximate time of filling the three locks referred to may be of interest. The data as far as available are given in Table 6.

The Sault installation gives a value for the coefficient of flow in culverts of 0.75 for six valves; the value for the Panama locks is given as 0.78. From such data as are given in the paper, it is inferred that the coefficient for Lake Washington is about 0.70.

The more rapid filling of the Pedro Miguel Lock seems to be due to the ample port area, since the culvert area has about the same relation to the chamber as in the Sault lock. On the other hand, the flow into the Lake Washington Lock appears to be throttled by the culvert area since the relation of ports to culverts is somewhat greater than at the Sault although, less than at Pedro Miguel. This does not indicate, however, that the culverts should have been larger, as the rate of filling would seem to be entirely satisfactory for the traffic concerned.

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PRECISE WEIR MEASUREMENTS

Discussion*

BY MESSRS. H. W. KING AND FLOYD A. NAGLER

H. W. KING,† M. AM. SOC. C. E. (by letter).‡—The authors have made a most valuable contribution to the subject of measuring flowing water by means of weirs. Their experiments explain possible causes of discrepancy in the results of earlier investigators and show quite clearly some of the conditions that affect the discharge over weirs. The most important of these conditions appear to be: (1) Sharpness of crest; (2) degree of roughness of face of weir; and (3) distribution of velocities in channel of approach. Although it has been known for some years that all these factors affect the discharge over thin-edged weirs their importance had not before been pointed out so clearly.

The new experiments indicate that opinions regarding the accuracy of weir measurements must be modified and also that the operations of measurement are more complicated than has heretofore been supposed. It still appears that the weir is a useful device for measuring water where a probable error of 2 to 5% is permissible. The statement§ of the authors that:

"The use of such a weir properly built, of good head measurements and good auxiliary velocity measurements in the channel of approach, should insure discharge values within 2% of the truth and in most cases within 1 per cent."

is undoubtedly a fair conclusion. The standard requirements of a sharp crest and smooth up-stream face, however, are not easy to obtain and maintain and the additional requirement of measuring velocities in the channel of approach takes much from the usefulness of what has heretofore been considered a simple means of measuring water.

* This discussion (of the paper by Ernest W. Schoder, M. Am. Soc. C. E., and the late Kenneth B. Turner, Esq., published in September, 1927, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Prof. of Hydr. Eng., Univ. of Michigan, Ann Arbor, Mich.

‡ Received by the Secretary, November 17, 1927.

§ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1400.

The new experimental data consistently give smaller discharges than were obtained by Bazin, and thus, in general, conform to the results of Francis, Fteley and Stearns, and Rehbock. The evidence appears to be overwhelmingly in favor of the smaller values. In this connection, however, it should be noted that Nagler's experiments,* which were performed on a weir, duplicating Bazin's standard weir, agreed substantially with the results obtained by Bazin. Although the discharges were not measured volumetrically, they were measured in a manner and under conditions such that their accuracy can hardly be doubted. It seems more likely that in duplicating approximately the conditions of the two sets of experiments the discharges obtained in each case were substantially the same.

In the discussion† of Mr. Nagler's paper, the late Frederic P. Stearns, Past-President, Am. Soc. C. E., makes the following statement:

"In view of these large differences in the results of carefully conducted weir experiments, two conclusions seem to be warranted:

(1) That in the present state of the art the weir is not an accurate instrument for the measurement of water;
(2) That the art should be developed so that the causes of the variation in results may be discovered and eliminated."

The first of these conclusions holds to-day. The later experiments only raise the query as to whether it is worth while to follow the suggestion contained in Mr. Stearns' second conclusion, or whether it will be more profitable in the end to let the sharp-crested weir rest temporarily and take up the investigation of other more promising means of measuring flowing water.

There is undoubtedly great need for some device of the weir or "obstruction" type for measuring discharges in open channals. The question arises as to whether this need is best met by the sharp-crested weir, or whether some other form of weir or obstruction in the channel can be designed for which the discharge will not be so greatly affected by minute variations in standard conditions. This is a field worthy of thorough investigation. In the past, many engineers appear to have accepted the sharp-crested weir as a panacea, and it has been investigated to the exclusion of other devices of a similar character.

The Society's Special Committee on Irrigation Hydraulics is investigating methods of measuring water in open channels, and is making a special study of devices for producing flow at critical depth. No recommendations as to a standard design have yet been made. It appears, however, from the work of the Committee to date‡ that discharges can be measured with great accuracy by a proper utilization of the critical-depth principle.

The discharge over a broad-crested weir occurs at critical depth. If the up-stream edge of such a weir, Fig. 50(*a*), and the up-stream edges of the ends of the weir are well rounded so as to prevent contraction, the discharge

* "Verification of the Bazin Weir Formula by Hydro-Chemical Gaugings," by Floyd A. Nagler, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 105.

† *Transactions*, Am. Soc. C. E., Vol., LXXXIII (1919–20), p. 172.

‡ "The Improved Venturi Flume," by Ralph L. Parshall, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 841; particularly discussions by Julian Hinds and J. C. Stevens, Members, Am. Soc. C. E., pp. 859 and 872.

will be given very nearly by the formula,

$$Q = 3.087\ L\ H^{\frac{3}{2}}$$

This formula has the inherent objection, in common with formulas for sharp-crested weirs, that the head, H, must be corrected for velocity of approach and that this correction is affected by the distribution of velocities in the channel of approach. It has advantages, however, over the sharp-crested weir in not requiring a sharp crest which is difficult to maintain and in being much less sensitive to conditions affecting roughness of crest.

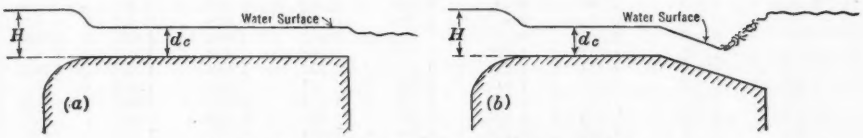

FIG. 50.—TYPES OF BROAD CRESTED WEIRS.

If the weir crest, Fig. 50(a), has a slope a little greater than is required to overcome the loss of head due to friction for discharge at critical depth, the water should flow at very nearly the critical depth, d_c, over the entire surface of the weir. Near the up-stream edge of the weir, just below the place where the drop in water surface occurs, the depth should be the critical depth within the limits in accuracy of measurement. The discharge is then given by the theoretical formula,

$$Q = L \sqrt{g}\ d_c^{\frac{3}{2}}$$

This formula is independent of conditions affecting velocity of approach and appears to have many advantages. It still lacks a thorough experimental test.

The broad-crested weir has a distinct advantage in that the discharge is not affected when the water surface on the down-stream side is a distance, d_c, or less, above the weir crest. A still greater submergence is permissible if the down-stream portion of the crest is given a steep slope, Fig. 50(b). This permits the water to obtain a velocity greater than the velocity at critical depth, and a hydraulic jump results. The loss of head in a low jump is comparatively small, and by proper design most of the energy of the water can be converted into static head.

The utilization of the critical-depth principle gives promise of providing a simple, accurate, and comparatively inexpensive method of measuring water. The Special Committee on Irrigation Hydraulics is to be congratulated on the work it has already done, and it is to be hoped that the investigation will be carried to the point where accurate coefficients will be available, and where it will be possible to make recommendations as to standard settings and methods.

An unpublished thesis, by C. J. McLean, Assoc. M. Am. Soc. C. E., submitted in connection with graduate work at the University of Michigan, contains the results of a series of experiments for determining the variation in head measured at different places in the channel leading up to a weir. A summary of Mr. McLean's experiments is contained in Table 56. These experi-

ments supplement similar experiments by Fteley and Stearns,* and show clearly the existence of the "angle of pressure".

TABLE 56.—GAUGE READINGS, IN FEET, SHOWING ANGLE OF PRESSURE ABOVE WEIR.

Gauge No. 1.	Gauge No. 2.	Gauge No. 3.	Gauge No. 4.	Gauge No. 5.	Gauge No. 6.	Gauge No. 7.	Gauge No. 8.	Gauge No. 9.
0.145	0.144	0.145	0.145	0.145	0.144	0.144	0.144	0.145
0.301	0.299	0.301	0.299	0.300	0.300	0.300	0.300	0.300
0.436	0.433	0.436	0.434	0.435	0.435	0.436	0.434	0.436
0.620	0.614	0.619	0.613	0.618	0.617	0.618	0.617	0.618
1.007	0.990	1.005	0.988	1.004	1.000	1.004	1.001	1.002
1.316	1.286	1.311	1.291	1.315	1.308	1.312	1.309	1.310
1.371	1.342	1.364	1.327	1.366	1.357	1.368	1.358	1.361
1.443	1.406	1.437	1.412	1.441	1.431	1.437	1.433	1.435
1.608	1.573	1.603	1.568	1.606	1.593	1.602	1.596	1.599
1.692	1.658	1.680	1.643	1.686	1.671	1.680	1.675	1.677
1.806	1.763	1.797	1.754	1.801	1.784	1.794	1.788	1.791
2.058	2.004	2.051	1.998	2.053	2.031	2.047	2.038	2.042
2.239	2.180	2.235	2.179	2.234	2.206	2.229	2.217	2.223
2.438	2.382	2.438	2.379	2.434	2.403	2.431	2.416	2.426
2.586	2.532	2.581	2.503	2.580	2.543	2.573	2.555	2.566
2.769	2.693	2.757	2.641	2.754	2.711	2.744	2.724	2.738
2.989	2.908	2.980	2.878	2.980	2.932	2.960	2.943	2.954
3.240	3.144	3.224	3.111	3.222	3.165	3.197	3.177	3.192
3.507	3.403	3.508	3.386	3.497	3.430	3.475	3.448	3.464
3.709	3.626	3.695	3.581	3.706	3.629	3.697	3.648	3.664

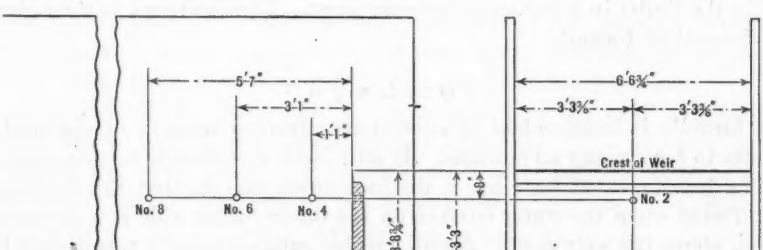

FIG. 51.—LOCATION OF CONNECTIONS TO HOOK-GAUGES.

The experiments were performed in the experimental flume at the University of Michigan, which is fully described in Mr. Nagler's paper. Heads were measured with hook-gauges that gave a range in reading of 4 ft. Openings were made in the face of the weir and on one side of the channel, at places selected for measuring heads. In these openings short pipes, 2 in. in diameter, were placed so as to be flush with the inner surface of the channel. The pipes were connected by a ¾-in. rubber hose to 5-gal. galvanized-iron pails. The pails were raised and lowered to positions that were convenient for

* *Transactions*, Am. Soc. C. E., Vol. XII (1883), p. 1, and Vol. LXXXIII (1919–20), p. 174.

reading the gauges. Nine connections were made at the positions indicated in Fig. 51. The location of the connection for Gauge No. 9 corresponds to that used by Bazin. Gauges Nos. 7 and 8 are connected at points corresponding approximately to points used, respectively, by Francis and Fteley and Stearns.

FLOYD A. NAGLER,* M. AM. SOC. C. E. (by letter).†—The outstanding contribution to the knowledge of the hydraulics of weirs made by this excellent paper exists in the investigation of the effect of irregular channel velocities on the quantity of water flowing over the crest. Some of the divergence between the results of standard experiments could no doubt be attributed to such a cause if the precise knowledge of the velocity conditions in the approach channel were available. Certainly, far different conditions existed in the long unobstructed channel leading to the Bazin weir than in the short, wide, and turbulent channel approaching the Fteley and Stearns 19-ft. weir, the actual current velocities of which were only partly described by the results of the current-meter measurements recorded with their experiments. The writer believes that future experimenters will use extra caution in securing a satisfactory distribution of flow in the approach channel. This distribution of flow will be examined by preliminary velocity measurements, and the discharge will be determined by some standard weir formula rather than by methods allowing a poor distribution of flow to exist which would require the measurement of current velocities for every observed head on the weir in order to solve for discharge in the authors' formula.

The experiments cited by the authors on the effect of rounding the weir crest and a roughening of the weir plate are also important, and are in good agreement with investigations which have been conducted since 1923 at the hydraulic laboratory of the State University of Iowa.

The Iowa investigations were performed on a sharp-crested weir, 2 ft. long and 3 ft. high. The crest of the weir plate was made of brass with a sharp and well maintained edge. The procedure followed by James Barr was used in roughening the weir plate by coating it with sand of several degrees of fineness. Care was exercised to file the crest after applying each coat so as to leave an irregular but sharply defined up-stream edge on the weir. How effective the roughening near the crest was in comparison with that some distance below the crest, was tested by stripping the roughened coat from the plate to various distances below the edge of the plate.

The appearance of the weir plate as prepared for several of the experiments is shown in Figs. 52 and 53. The weir in Fig. 52 was covered with coarse sand that was retained on a No. 8 standard Tyler sieve, but passed the No. 4 sieve. The weir in Fig. 53 is studded with hardware.

While these experiments were performed primarily to determine the effect of the roughening on the profile of and velocities within the nappe, volumetric measurements were made to compare the discharge of the weir with its flow when its crest was sharp and the weir plate smooth. A summary of the results of these measurements are given in Table 57.

* Prof. of Hydr. Eng., Univ. of Iowa, Iowa City, Iowa.
† Received by the Secretary, November 17, 1927.

It is interesting to note that the roughening of the weir plate near to the crest was much more effective in increasing the discharge than the roughening more than ½ in. away, and that roughening the wooden bulkhead by heavy hardware caused no alteration in discharge.

TABLE 57.—PERCENTAGE OF INCREASE IN DISCHARGE DUE TO ROUGHENING OF WEIR PLATE.

Series No.	Condition of weir plate.	Head on Weir.			
		0.2 ft.	0.3 ft.	0.4 ft.	0.5 ft.
	Coarse sand:				
2	(Retained on No. 8 standard sieve and passing No. 4) to crest	6.5	6.0	6.0	4.7
3	To crest with bulkhead studded	6.5	6.0	6.0	4.7
4	To ¼ in. from crest	0.2	1.0	2.3	2.9
	Medium sand:				
5	(Retained on No. 28 sieve and passing No. 14) to crest	4.0	2.4	2.0	1.5
6	To 1/16 in. from crest	1.2	1.5	1.5	1.5
7	To ⅛ in. from crest	0.7	0.7	0.6	1.5
9	To ½ in. from crest	0.3	0.3	0.0	0.0

Experiments performed by the writer* have demonstrated that the Bazin formula may be used in computing discharges for heads even higher than those included in the writer's experiments, securing results much nearer the truth than those obtained by the solution of the Francis formula. The authors' experiments seem to substantiate this conclusion. Under low heads the results obtained with the Bazin formula are higher than those obtained by most other experimenters, but the explanation of the authors† seems somewhat presumptuous and misleading with regard to the true condition of the Bazin weir. After considerable study and investigation the writer has come to the conclusion that the Bazin weir plate was unquestionably sharp and smooth, and for the following reasons:

(1) Hydraulic science has never known a more careful and better trained observer than M. Bazin, and it appears almost unthinkable that a man who had spent a large part of his life carefully studying into every small detail of the phenomena of the nappe passing over the standard weir, should have carelessly allowed the weir plate to become rough and the edge other than sharp. When Bazin used the words "arête vive",‡ an expression was used such as would correctly be applied to the sharp and well-defined knife-edge of a fine chemical balance, and it has been correctly applied to the weir in translating it as a sharp edge. Also, the words, "bien dressee",§ applied to the weir plate indicate that considerable attention was given to the condition of the face of the weir plate itself.

(2) A few years ago the writer raised the question of the condition of the Bazin weir plate in some correspondence with M. Hegly, who it will be

* *Transactions*, Am. Soc. C. E., Vol. LXXXIII (1919–20), p. 105.

† *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1449, and elsewhere.

‡ *Annales des Ponts et Chaussées*, October, 1888, pp. 395, 400, and 444.

§ *Loc. cit.*, p. 400.

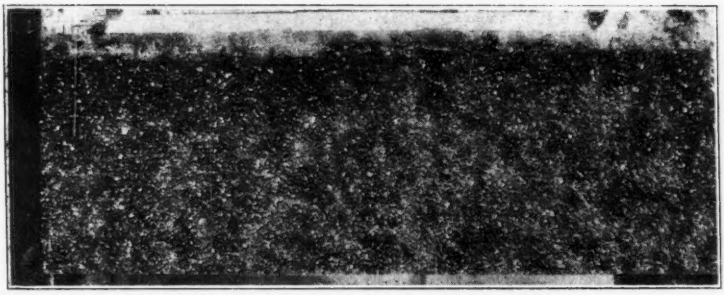

FIG. 52.—WEIR PLATE IN SERIES 2.

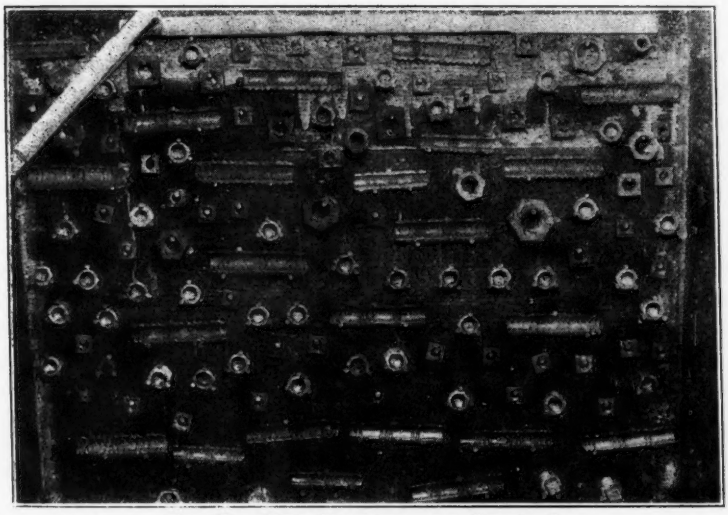

FIG. 53.—WEIR BULKHEAD FOR SERIES 3.

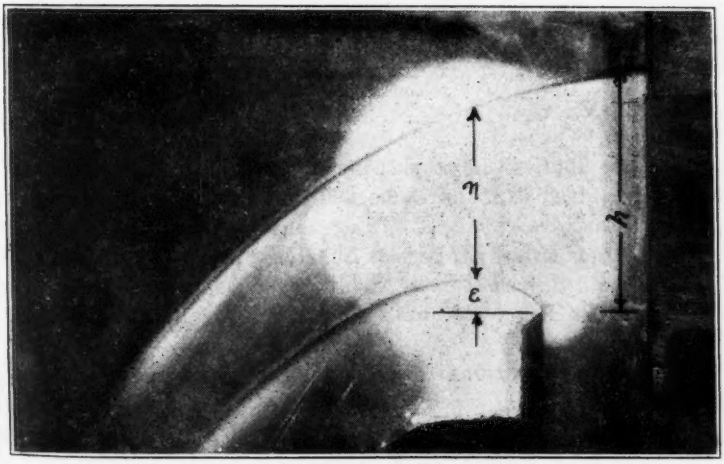

FIG. 54.—PROFILE OF WEIR NAPPE.

recalled was associated with Bazin in his weir experiments.* M. Hegly made reply in no uncertain terms, which the writer has translated as follows:

"No special photographs were taken of the crest of the standard weir used in the Bazin experiments. I only know he gave much attention to the question of assuring himself that the edge, where the under surface of the nappe escaped, was level and corresponded to a straight line. And, several times, the crest, that is to say, the plate of wrought iron 7 mm. thick, which rested on top of the wooden bulkhead, was removed and laid on the side of the experimental canal, in order that he might by touch as well as by sight determine the sharpness of the up-stream edge of the plate. He never permitted any doubt on this question.

"From time to time he cleaned the top and up-stream face of the weir plate in order to remove small deposits of algæ and rust which might have affected the accuracy of the crest."

These statements of M. Hegly leave no doubt as to the condition of the Bazin weir crest. In fact, Bazin was probably more meticulous in maintaining a sharp edge on the weir crest than any of the other early investigators.

(3) Bazin's measurements on the profile of the lower face of the weir nappe unknowingly demonstrate both the sharpness of the crest and the smoothness of the weir plate.

Measurements of the weir nappe made in the hydraulic laboratory of the State University of Iowa, have shown that the value of ϵ (Fig. 54) varies not only with the height of the weir, but with both the degree of sharpness of the crest and also the roughness of the up-stream face of the weir. A summary of the results of these experiments is given in Table 58. The weir was 3 ft. high and 2 ft. long.

TABLE 58.—Maximum Elevation of Lower Surface of Weir Nappe.

Series.	Condition of weir.	Value of $\frac{\epsilon}{h}$.
1	Standard sharp crest	0.108
5	Sand to crest	0.092
6	Sand to $\frac{1}{16}$ in. from crest	0.097
7	Sand to $\frac{1}{8}$ in. from crest	0.099
8	Sand to $\frac{1}{4}$ in. from crest	0.102
9	Sand to $\frac{1}{2}$ in. from crest	0.103

Bazin obtained an average value of ϵ equal to 0.112 on a weir 3.72 ft. high, and an average value of 0.099 on a weir 1.15 ft. high.† Arithmetical interpolation between the values for the Bazin weirs gives a value for ϵ of 0.108 for a weir 3 ft. high, similar to that of the writer, but with a crest condition duplicating that of Bazin. This value is exactly that obtained on the weir at the University of Iowa, which the writer can affirm was thoroughly sharp.

These facts make it appear conclusive that the Bazin weir had a truly sharp edge, and it is regretted that the authors' ingenious explanation of the divergence of the Bazin low-head experiments seems untenable.

* *Annales des Ponts et Chaussées*, October, 1888, p. 448.

† *Loc. cit.*, 1890, p. 14.

The type of formula proposed by S. M. Woodward, M. Am. Soc. C. E.,* has many practical advantages in solution, which make it far more usable than that proposed by the authors; and if this formula had been checked against the original data instead of against the authors' Equation (*D*),† it probably would be found to represent these data with equal accuracy.

The authors are to be commended for the completion of the colossal task of assembling such a large mass of experimental data taken by many observers through a long period of years, and it is remarkable that the inconsistencies are as few as they appear to be.

For work in preparing the tables in this discussion, acknowledgment is due to Messrs. J. W. Howe, E. T. Schuleen, and Glenn N. Cox, graduate students at the State University of Iowa.

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1436.
† *Loc. cit.*, p. 1434.

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PAPERS AND DISCUSSIONS

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BASIC INFORMATION NEEDED FOR A REGIONAL PLAN

Discussion*

By Harland Bartholomew, M. Am. Soc. C. E.

Harland Bartholomew,† M. Am. Soc. C. E. (by letter).‡—Regional planning is the logical and inevitable outgrowth of city planning. While an ever-increasing application of the principles of city planning may be expected in towns and cities, there will also be increasing need for the application of definite technique and scientific principles in regional planning. Difficult as is the preparation, adoption, and execution of a genuinely comprehensive city plan, of far greater magnitude and complexity are the difficulties of regional planning. In this initial stage of work it would be most unfortunate to attempt to specify means and methods too closely. This is probably the most complicated of the structures created by Man.

It is most fortunate that the initial undertaking of regional planning in America by the Committee on Regional Plan of New York and Its Environs has proceeded without some preconceived objective in view. The divisions of the work, that is, physical, economic, social, legal, and financial, are each important and necessary. The extent to which basic information could and should be secured is as yet impossible to estimate. It will be limited in each particular case by the nature and comprehensiveness of the project and the funds at hand. Unfortunately, there will probably be few projects undertaken with the breadth of view and with the resources that have been available to the New York organization.

It is quite probable that subsequent regional plans will sacrifice comprehensiveness for specific accomplishment in such fields as streets and highways, in transportation, or in parks. This will be productive of practical knowledge and results in individual instances, the summation of which will

* Discussion on the paper by Harold M. Lewis, M. Am. Soc. C. E., continued from December, 1927, *Proceedings*.

† (Harland Bartholomew and Associates), St. Louis, Mo

‡ Received by the Secretary, November 9, 1927.

develop definite technique. It will increase current knowledge of the character of basic information of all kinds needed for a complete regional plan.

Mr. Lewis' paper is an excellent summary of the present knowledge of the subject of office technique growing out of his experience in the New York work. If one were to single out a specific subject for discussion, it might be the question of a base map. The inadequacy of existing maps of particular regions is complete. They just do not exist. In Washington, D. C., as in New York, for instance, it was found necessary to resort to the sheets of the U. S. Geological Survey and with the co-operation of the latter to bring existing information up to date and at a workable scale. The 2 000-ft. scale map used in the New York studies is very desirable and will undoubtedly become more or less standard. The U. S. Geological Survey has just completed a new survey of the Los Angeles, Calif., region at this (2 000-ft.) scale, which is exceptionally good. Where the Geological Survey sheets are not available, or are not up to date, there is an initial and well-nigh insuperable handicap on the work of starting a regional plan. There is even an astounding lack of good topographic maps in cities. One of the by-products of regional planning, as of city planning, will be more and improved topographic maps.

The scope of this subject is inexhaustible. Mr. Lewis includes in his "Physical Survey,"* "Population—distribution, density and rates of growth", and also mentions that special inquiry has been made of "Standards of Population and Housing Density".* This information is of particular importance and should be augmented considerably. To what extent it has been considered in the New York study Mr .Lewis does not indicate, but this will doubtless be found in present or future reports of the Committee. Here, the economic and social surveys overlap the physical. Ultimately, regional planners are going to find out how widely they can disperse a given population (or congest it for that matter) and produce a condition that is both socially desirable and economically sound. This is "basic information" of the greatest importance.

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1507.

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PAPERS AND DISCUSSIONS

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FORECAST: THE REGIONAL COMMUNITY OF THE FUTURE

Discussion*

By Morris Knowles, M. Am. Soc. C. E.

Morris Knowles,† M. Am. Soc. C. E. (by letter).‡—The regional community is an interesting subject, and the author's forecast of future urban regions may well arouse enthusiasm and stimulate discussion. Comparison with present-day development serves only to magnify the problems; those of construction as well as those of administration. However, there is ample experience now, from the results of not anticipating and planning for the future in the building of American cities, to provide a background of helpful consideration. The measure of success in the future will depend upon how well this experience is to be utilized in planning and in effecting governmental and administrative control. The two are inseparable. If improvement is to be realized, first it must be conceived, then designed, that is, planned; and then comes the necessity of practical administration to convert the plans into actual accomplishments. A look into the future, as to the probable type of new governmental agencies, necessary to secure results is the burden of this discussion.

Aside from the inherent social instinct, concentrated community life has come about naturally because of the advantages that accrue from the pooling of human endeavors. Different forms of government have been evolved and machinery has been set up for the construction of public facilities and the administration of public affairs. Through these agencies, the needs of the community have been supplied; water and sewerage systems have been planned and built; streets have been laid out and paved; police and fire protection methods have been developed; and all the other conveniences and comforts of modern life.

* Discussion on the paper by Thomas Adams, Esq., continued from December, 1927, *Proceedings*.

† Pres. and Chf. Engr., Morris Knowles, Inc., Pittsburgh, Pa.

‡ Received by the Secretary, November 3, 1927.

In accomplishing these results, some extravagance and waste have been inevitable; more often than not the needs of the future have been under-estimated in the construction of public works; mistakes have been made which have proved costly or impossible to correct; streets have been laid out with insufficient width; there has been too much crowding of the land in building operations; water and sewerage systems, as constructed, have proved inadequate and many other deficiencies have been found to exist.

Early Development of Comprehensive Planning.—Out of this has come the modern conception of comprehensive city planning, designed to correct some of the mistakes of present city activities and to create an ideal toward which to build in the future. The subject is quite new, its development as a definite, conscious ideal having been within the past 10 or 12 years. However, it was soon realized that many of the problems of such planning would be of a regional character and necessarily must include the suburban, satellite, and semi-rural districts surrounding the city and, in many instances, neighboring communities.

Regional planning as a basis for community activities and the problems involved in such study were presented by Mr. Adams—then Housing and Town Planning Adviser of the Canadian Government—at the meeting of the National Conference on City Planning at Niagara Falls and Buffalo, N. Y., in 1919. Since then, much study has been given to the subject and to practical methods of adapting well-intentioned theories to actual practice. Various methods and agencies have been suggested, some of which have been tried and found to be partly successful; but there is still a realization that, while much educative effort has been developed, the correct solution has not yet been presented. The process of evolution in the consideration of the problem has been somewhat as follows:

(1) The need of a regional conception of utility development has been apparent.
(2) Realization that single unit utilities, however well planned and executed, sometimes fail to co-ordinate with each other, has brought about a recognition of the necessity of comprehensive planning.
(3) Failure to find ready means to carry well-intentioned plans into execution, has shown a need of an adequate agency to finance and execute these comprehensive plans.
(4) Thus has come about a recognition that a new governmental attitude is necessary.

It is the writer's purpose to amplify and explain some of this development of thought in this discussion and suggest a type of government for the regional community.

Utilities—A Basis for Regional Planning.—Probably the first recognition of regional planning originated through the need of co-operation in the development of public utilities, where the service was common among municipal units arbitrarily separated by political lines. The city, in addition to its legislative, taxation, educational, and police power functions, is an industrial plant and its government an organization for carrying on the affairs of

municipal management. It is apparent, therefore, that public works and utilities are of first importance in considering co-operative action.

Where municipalities lie so closely together as to be practically continuous, many municipal functions can be most efficiently performed under a single management. Such services as water supply, drainage, sewerage, main thoroughfares, general park developments, and regional planning are obviously accomplished best under unified control. Only in this way can haphazard development and duplication be avoided. These problems cannot be correctly solved within man-made political boundaries, but must be treated as watershed and area problems for a district having similar conditions. Finally, it is generally true that economy, both in construction and operation, is obtained by collective action.

Entirely apart from these engineering and economic considerations, however, there are sound reasons of political morality for the co-operative solution of such problems. No one "liveth to himself" and urbans and ruralists are all "members one of another". The suburbanite, working and buying in the city, is as dependent on the city's service as the city dweller who trades and visits in the environs, is on the facilities of the outdoor area. Mutual dependence involves mutual obligations and there is mutual advantage in sharing alike the common benefits in costs.

The earliest method devised to bring about concerted action among neighboring communities has been through organization of metropolitan districts for specific purposes. However, co-operative community development involves consideration of a much broader field of activity.

Problems of Regional Planning.—Because of the influence and effect of one group upon another in such matters as fire and police protection, water supply, sewage disposal, and prevention of the spread of disease, it is necessary to think in terms of the region, rather than of the separate units which enter into the combination. For example, one community may have an improved and sanitary water system, while a neighboring city may furnish an unwholesome supply. Similarly, the discharge of waste from one community may pollute the stream from which another draws its water; or, in the control of the milk supply, unified action is necessary if it is to be effective in accomplishing the desired results. These illustrations, while they refer only in a meager way to the many problems to be considered, include examples both of control and of planning.

Area Affected.—The region, with which such planning is concerned, might be defined as an area which, on account of the interdependence of the various communities therein, may be considered as a unit in physical and cultural development. Such an area might include several important cities, perhaps approximately of the same size, all having common interests and mutually dependent one upon another; or it might comprise a metropolis and its neighboring suburbs and satellite towns.

The latter type illustrates a most interesting fact about the modern growth of great cities; that is, the tendency to spill their populations out over a large area rather than to develop progressively outward from the center

as was formerly the case. Thus, separate communities spring up all along the transit lines and at other favored spots, adjacent to the city, but outside its corporate limits. Mercantile establishments frequently find it profitable to move farther from the heart of the city; and high-class shopping districts, including theatres and hotels, are springing up in the outlying communities. Industries also find it to their advantage to choose sites farther from the center, where land values are lower and where there is room for new and modern housing developments which will be attractive to workmen.

Importance of Common Problems.—The planning and development of the region, therefore, including all the various towns and communities tributary to the parent city, offer far greater opportunities than in the case of the old-time progressive expansion of the city from the center outward; but, at the same time, new problems are introduced which were not present when one political boundary line enclosed the entire urban community, and growth beyond it was in concentric rings. Consequently, as the satellite communities grow, regional problems become grievous and separate municipalities find it difficult to arrive at any solution unless one helps the other, or joins with the other, and all agree on a common plan.

Advantages of Co-Operation.—Even superficial thought on the subject will convince one, therefore, that resulting benefits may be expected if a large number of political units—separated only by imaginary political boundary lines, but forming as a whole one unit as to economic, industrial, commercial, and social life—are united by some plan that makes concerted action possible in the conduct of public affairs. Some of the results of practical co-operation to be expected, are conservation of resources, economy in the development and construction of public facilities, and greater efficiency in governmental administration.

However, reference to some actual accomplishments of consolidation may be of interest and will afford convincing argument as to the advantages. The following is quoted from a paper* by Professor Chester B. Maxey, of the Western Reserve University, referring to the formation of Greater Philadelphia, during the Mid-Nineteenth Century:

"The immediate results of the Consolidation and Annexation were unquestionably beneficial. For many years Philadelphia and her environs had been disgraced by civil turbulence almost impossible to prevent or control. * * * These disturbances and disorders quickly disappeared when unified local government went into operation. * * * Other beneficial results of consolidation were the development of a comprehensive system of water supply and sewage disposal to replace the separate systems of the former independent municipalities; the establishment of a metropolitan park system; and a marked increase in the assessed valuation of property throughout the metropolitan area."

Similar illustrations could be given, but there is no need of repetition.

Regional Planning Commissions.—Naturally, in order to accomplish effective results in regional planning, there must be some form of centralized

* "Political Integration of Metropolitan Communities," *National Municipal Review*, August, 1922.

control over the major functions of the separate communities, in matters of common concern. Various methods of accomplishing such control have been considered and some have been tested, with more or less success. The Standard City Planning Enabling Act, as prepared by the Advisory Committee on City Planning and Zoning of the U. S. Department of Commerce, and published in preliminary form in February, 1927, makes provision for the establishment of regions and for the creation of regional planning commissions.

One method of creating the regions is that of having the Governor determine the boundaries and appoint the members of the regional planning commission. This would be done on request of the planning commission of any municipality, the county commissioners of any county, or upon petition of a certain number of citizens. An alternative method also is described, which contemplates the creation of the regional planning commissions on the initiative of one or more local planning commissions and the county commissioners of the counties involved. In the latter instance these agencies, by mutual agreement, would decide on the boundaries of the region and would determine the relative share of the cost of the work to be borne by the respective municipalities and counties and the size and method of selection of the regional planning commission.

The organization of the commission and the powers and duties as proposed, would be similar to those of a city planning commission. Provision also has been made in the Act for certification of the regional plan to the various interested agencies within the region; for the adoption of the plan by municipalities; and for the legal status of the regional plan after its adoption.

This, so far as it goes, is a beginning, but an equally if not more important consideration is the creation of centralized administrative and executive control. In the city, after the planning commission has developed a general city plan, the existing departments of the municipal government are available for the handling of the detailed designs and for the construction of improvements; but in regional planning there is no form of centralized government available to bring about concerted action in matters of common interest, nor to initiate, execute, and administer the regional projects that have been conceived. Commissions or boards may be appointed of course to carry out some detail of the general plan, but the co-ordination of such specific features with other improvements proposed by the regional planning commission may be forgotten. Moreover, such efforts refer more particularly to construction problems and do not take into consideration the consolidation of the community in other important matters of public welfare. The real solution lies in the development of some form of definite governmental authority which will secure the benefit of combining the resources of the separate political units in matters of regional character; but which will guarantee to such separate units local control in affairs of purely local concern.

Methods of Organization.—While the details of organization and administration may differ, as proposed in any particular instance, the various plans tried, or proposed throughout the country, may be grouped under one of three headings, as far as general principles are concerned. Stated briefly, these are as follows:

1.—Annexation of Adjoining Territory.
2.—Metropolitan Districts for Specific Purposes.
3.—A Federated City Plan of Government.

Annexation.—This is the oldest method of consolidation and the one that has been resorted to in the growth of practically every city. It includes the actual incorporation of adjacent territory into the city, and complete centralization of authority with respect to public works, public safety, legislation, taxation, and all other municipal functions. The result may be brought about by mutual agreement between the city and the annexed suburbs; or, under some forms of legislation, by the sovereign power of the State, or by a majority of all the voters in the territory affected, regardless of the wishes of the voters of the suburbs to be annexed.

Voluntary annexation of neighboring suburban towns in the larger cities often may be difficult to accomplish. This is not difficult to understand, for the people of smaller communities are vastly more homogeneous in regard to their concerted actions than people of the wards of the city. They feel more at home and more friendly with each other, because the people in city communities are changing so rapidly that permanency of abode does not exist to a sufficient degree to promote acquaintance and friendliness. Pride in one's own small community is natural and to be encouraged, and it is not to be expected that residents in such places will voluntarily delegate to a larger municipal authority, more or less removed, powers which have to do with purely local affairs and which now are thought to be capably managed by officials whom they know and who, perhaps, are neighbors.

However, if the need be sufficiently urgent, annexation may be carried out and has been, in several important instances, sometimes voluntarily and sometimes forcefully or involuntarily. The difficulty lies, not in any consitutional limitations in most States in applying the latter method, for the municipality, of course, is simply a creature of the State and subject to whatever laws the Legislature may formulate; nor is the only objection the fact that the principle of home rule would be discarded entirely.

The fact is that annexation, whether voluntary or otherwise, does not solve the problem. There is destroyed that very desirable quality of people wishing to do things for themselves; pride in small local affairs is lessened; the older, well-developed portions of the city are burdened with the cost of improvements in outlying sections; and the tendency is again toward centralization and congestion. Thus, the problems of municipal government become increasingly complex and difficult, leading too often to misgovernment and mismanagement.

Metropolitan Districts.—This method approaches the solving of regional problems through metropolitan commissions, each charged with some particular type of public works or activity, and functioning within a region known as a metropolitan district. This plan has had its greatest development in London, England, through the activities of the London County Council, and has been copied, to some extent, in this country. For example, around Boston, Mass., there have been various metropolitan commissions since about 1890, and

some of the best examples in the United States: First, a Sewerage Commission, covering Boston and 15 towns within a radius of 15 miles or more of the State House; then, in 1895-96, the development of the Metropolitan Water Commission; and, about the same time, the Metropolitan Park Commission. Some time later, the Metropolitan Police was organized.

These various commissions have cared wonderfully well for certain types of public improvements, and for their proper design and efficient operation. It should be remembered, however, that the operation of these Boston commissions is entirely in the hands of the State through commissions appointed by the Governor. This paternalistic authority of the State over the communities is really more exacting and detailed than a municipality would be likely to have exercised. The local communities have nothing to say; they do not vote for appropriations or commissioners; they do not get together; the State does it all and then charges the cost to each.

Recently, having in mind that these things were not operating as well as if they were more unified, the authorities resolved to join the three commissions having to do with water supply, sewerage, and parks, into one Metropolitan Commission. This has helped, because it means that, when water-works are planned, there will be some consideration given to drainage and sewerage and parks, and *vice versa*. It readily can be seen, however, that many other features of the metropolitan area also are affected. There is still a lack of co-ordination of the various governmental affairs within the region, which must be considered. Conflicting action rather than harmony is produced. Only recently a retrogression has occurred in the creation of a board for the development of a new, distant source of water supply.

There are many other metropolitan commissions functioning in American cities, most of them having to do with only one particular kind of public works. The Sanitary District of Chicago, for instance, comprises all of Cook County and some territory outside. Its Commissioners are elected by the people. Another example is the Miami Conservancy District. This is probably the largest area of its kind in the United States and is a somewhat special case. The District embraces all or parts of nine counties and includes the Cities of Dayton, Middletown, and Hamilton, Ohio. It was organized for the purpose of constructing and maintaining flood-prevention works, and its government rests with a Board of three, appointed by a Court, consisting of one Common Pleas Judge in each of the counties affected.

Another example is the Essex Border Utilities Commission, constituted by Special Act of the Ontario Parliament, in April, 1916. Its object has been the solution of problems affecting the whole of the border municipalities, as distinguished from those to be dealt with by any one of them in particular. These problems include sewerage, water supply system, health, parks, planning, etc. The Commission consists of nineteen members, eleven of whom are elected and eight are *ex officio*, all serving without remuneration.

While the metropolitan district plan is frequently adapted to the development of certain specific public improvements, there are certain faults and disadvantages to be considered. Sufficient co-ordination between the various

types of improvements is lacking; control is remote from the people directly affected; and there is a possibility of much confusion and wasted effort from the overlapping jurisdictions of a great number of such commissions if many activities are undertaken.

Federated City Plan of Government.—This is the type of organization that has been adopted by the Pennsylvania Commission for the Study of Consolidation of Municipalities, after several years' study of the various methods, and this type is proposed for Greater Pittsburgh.

The plan proposed is somewhat analogous to that of New York City, but there are important differences. While New York comprises five Boroughs—each one a large city in itself, and some of which even have regional characteristics—the Pittsburgh Plan will include 124 municipalities, most of them already distinct and separate communities, which will be preserved within the consolidated city.

While the plan that has been proposed was formulated to meet particular local conditions within Allegheny County, in Western Pennsylvania, its general principles may be considered applicable to the majority of similar situations; and an explanation of its more important features may be of general interest.

The Greater Pittsburgh Plan.—The area included in the Pittsburgh Plan coincides with that of Allegheny County. Within this area, in addition to the City of Pittsburgh, there are three cities of the third class, sixty-six boroughs, and fifty-four townships. The population included will approximate 1 500 000. One of the basic principles adopted by the Commission in its preliminary consideration of the problem, as set forth in its report to the Governor, was:

"That such a plan, to be made satisfactory, should give to the constituent and smaller municipalities, proper and reasonable control of their local affairs * * *. No government must become too remote from the people. The greater things should be handled in a greater way; the smaller and more intimate things in a smaller way, a local way—home rule by home people. Local officials should and must be close to the citizens for whom they function, giving service as desired, never remote from or unmindful of the constituents, or out of constant touch with them."

An amendment to the Constitution of the State was found necessary in order to enable the adoption of such a plan. It was passed first at the special session of the Legislature in 1926 and again at the regular session in 1927. It will be placed before the people of the State in the form of a referendum in November, 1929.

While the constitutional amendment is largely of an enabling nature, the general features to be observed in the plan have been incorporated in it. This was done to insure certainty of rights to the smaller communities. They include provisions for the general functions and powers of the consolidated city; a statement that local powers are to be reserved to constituent municipalities; and an outline of the method of framing and adopting the new charter.

The charter, under which the consolidated city will operate, will be authorized by the General Assembly and will provide for,

"* * * the consolidation of the county, poor districts, cities, boroughs and townships of the County of Allegheny and the offices thereof, into a con-

solidated city and county, with the constitutional and legal capacity of a municipal corporation, to be known as the City of Pittsburgh and * * * provide for a charter for its government".

This provision will give to the consolidated city, authority to deal with problems affecting any two or more municipalities, or the whole district. Among such problems may be mentioned water supply, including the planning of a general system, where found advisable; sewerage systems, particularly the consideration of main trunk lines and of disposal; transit lines, including the beginning of a rapid transit system to serve adequately the greater city; main thoroughfares, through highways, tunnels, subways, and bridges; and other public works of a regional character which should be accomplished by, or under, the direction of the consolidated city government.

Another of the provisions relating to powers conferred on the new government refers to zoning, making such procedure constitutional beyond any possible doubt within the Greater City. Another grants authority to the Commissioners of the Central Government to create special improvement districts for the financing and construction of certain works, such as drainage, sewage disposal, or garbage collection and disposal.

Certain debts of the constituent units, incurred for the construction of public works which would be transferred to the Greater City, will be assumed by the Central Government. In addition, the charter may provide,

"For the assessment of property for taxation, the levying and collection of taxes and the payment of the costs of any public improvement, either in whole or in part, by special assessment upon abutting and non-abutting property materially benefitted thereby, and for this purpose real estate so charged shall be classified as urban, suburban, and rural, and adjustment made in accordance with such classification".

This obviously is the proper method of assessment for improvements rather than that which places the burden of the entire cost on abutting properties by general taxation through bond issues.

A guaranty against forcible annexation is given to the separate communities in case the charter is adopted, but there is nothing to prevent two or more of the separate municipalities joining together voluntarily whenever they desire.

The local powers to be kept by the several cities, boroughs, and townships are definitely stated in the proposed amendment, and include the following:

1.—The constitutional and legal capacity of municipal corporations, except as limited in the charter.
2.—The power to levy and collect taxes for any lawful power of the separate municipalities.
3.—The power to acquire, own, construct, maintain, and operate public property, works, improvements, utilities or services dealing with only local needs.
4.—The power to maintain a local police and fire department. In some cases this may be supplemental to the department of the consolidated city.
5.—All other powers not specifically granted by the charter to the consolidated city.

Provision is made for the voluntary surrender of any of these powers to the consolidated city, by vote of the electors in one of the separate municipal divisions, and by acceptance in each instance of such delegation of power and responsibility by the Board of Commissioners of the consolidated city.

Those who have given most thought to the situation, feel that the Pittsburgh Plan will provide a practical solution for regional problems of planning and for construction and operation of regional public works. Much will depend on the type and details of the charter yet to be drawn. However, even if all the results as to governmental operation may be as anticipated, the working out of the details and the thorough and practical planning of the region will be essential to effective accomplishment.

Wisdom of Regional Planning.—Community planning is based on the principle of co-ordination. Public facilities always have been planned to some extent but usually as independent and separate projects, so that there was a lack of harmony in fitting them into the general community plan. This has been the deficiency that brought about a realization of the need of something different and that has been responsible for the development of the present-day art of community planning. The important difference lies in the co-ordinating of all improvement plans and fitting them together into an harmonious and unified scheme of general design whether it be for a village, a city, or a region.

Consequently, the planning of a regional utility, such as a water supply or a sewerage system, by a regional or metropolitan board may be contrasted with the separate planning of a similar utility for one of the constituent communities, by the city engineer or by a municipal commission. Efforts like the latter cannot be described as regional planning activities; real value can come only from a comprehensive consideration of all the problems that are of a regional character. Otherwise, the same difficulties that have become so manifest with the growth of urban communities will eventually be experienced as the larger regional area becomes more compactly developed.

The difficulties, at the present time, lie in the lack of governmental machinery for developing the regional plan, and after having developed it, in making the plan effective in its administration. Whatever the method may be, the three steps to be considered in the development of a regional community are: (*a*) The conception and creation of the form of consolidation; (*b*) the establishment of the boundaries of the region; and (*c*) the adequate planning of public facilities and services for this area.

The whole subject is so new that there are only a few precedents or experiments as illustrations of what to do or of what to avoid; but in all private and public endeavors, planning should precede execution. Therefore, whether the start be made through the creation of a regional planning commission, with the subsequent details of the regional municipal organization to be developed later; or whether the general plan of governmental consolidation be first conceived, and the planning done thereunder, perhaps is immaterial. In either event both are essential to complete success.

Finally, community planning, whether it be that which is concerned with a single municipality or whether it be sufficiently enlarged in scope to include

many diverse political entities, should be conceived to accomplish more than the economical construction of public utilities, or the development of more efficient forms of public control. The psychological result should be to create a public consciousness and appreciation of community problems, and this, in turn, should be used to advantage in promoting enthusiasm for a large and intelligent program for the whole community. The modern development of American cities, and the advent of the automobile, which has made possible the scattering of populations over large areas at no sacrifice of time in transportation, have re-acted to a lack of participating interest in the larger civic affairs by those who make their homes in the suburbs or in the neighboring countryside. The larger community interests, because of their remoteness, have become secondary to those of the immediate local neighborhoods; but regional endeavors should be a means of bringing about a closer bond of sympathy and helpfulness in the solution of the many problems that confront the larger urban developments of the present day. The Pittsburgh Plan of Municipal Consolidation is offered as an advanced idea in the hope that it will merit public discussion and point the way to making regional plans effective through the development of a widespread civic consciousness in governmental affairs over large areas having common problems.

AMERICAN SOCIETY OF CIVIL ENGINEERS
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PAPERS AND DISCUSSIONS

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HISTORIC REVIEW OF THE DEVELOPMENT OF SANITARY ENGINEERING IN THE UNITED STATES DURING THE PAST ONE HUNDRED AND FIFTY YEARS

A SYMPOSIUM

Discussion*

BY MESSRS. GEORGE S. WEBSTER, MORRIS KNOWLES, R. A. HART, L. L. HIDINGER, AND ROY N. TOWL.

GEORGE S. WEBSTER,† PAST-PRESIDENT, AM. SOC. C. E.—Relative to Mr. Eddy's paper, the speaker has been asked to discuss briefly the growth of the sewerage system in Philadelphia, Pa. Philadelphia was one of the first of the large cities in the United States to be laid out on a systematic plan. This was done in 1682 by Thomas Holme, Surveyor to William Penn, the first proprietor of the Commonwealth of Pennsylvania. In the development of these early plans, provision was made for drainage systems.

The first of these drainage channels was built about 100 years ago. They were built of bricks laid with dry inverts, that is, without mortar joints; were generally 7 or 8 ft. deep and were intended to carry off both surface and ground-water. No buildings were connected with them at that time. Later, many of these drainage channels became part of the combined system of sewers of the city. The first effort to determine the size of sewers for run-off was made by the Board of Aldermen and not by the City Engineer. The Aldermen decreed that all branch sewers should be 3 ft. in diameter.

In 1875, the speaker was in charge of a Field Corps of Engineers at the Centennial Exposition Grounds in Philadelphia and, under direction of those

* Discussion of the Symposium on Historic Review of the Development of Sanitary Engineering in the United States During the Past One Hundred and Fifty Years, continued from December, 1927, *Proceedings*.

† Cons. Engr., Philadelphia, Pa.

in authority, gave lines and grades, and supervised the construction of sanitary sewers discharging directly into the Schuylkill River above the dam and into the pool from which a part of the City's water supply was taken and furnished without treatment to the citizens. This resulted in typhoid fever in October and November, 1876—a serious epidemic in Philadelphia.

This instance indicates the state of the science of Sanitary Engineering at that date and emphasizes the great advancement that has been made in the last 50 years.

At the time of the Centennial Exposition in 1876, the late Rudolph Hering, M. Am. Soc. C. E., obtained from the Swiss Exhibit, a diagram showing the Ganguillet and Kutter curves for the flow of water in channels and the speaker made a tracing of it for the use of the City Engineering Department and for the Engineering Profession in general.

The first empirical formula of which the speaker has any knowledge, that was used for determining run-off when designing sewers in Philadelphia, was "one cubic foot per acre per second", which is equivalent to 1 in. of rainfall per hour, all reaching the sewer. This is not a bad rule for an engineer to use as a rough check on his assistants in calculating run-off for large sewers.

In 1880, the American Public Health Association sent Mr. Hering to Europe to investigate sewerage practice, and his report thereon marks one of the turning points in sewer design in this country as it placed such design on a scientific basis rather than by rule-of-thumb. He brought back with him the Burkli-Ziegler formula. This was thought to be the last word in determining the run-off for storm-water sewers. It was soon found, however, that in a city like Philadelphia, with street grades varying from 1 ft. in 1 000 ft. in the flat sections to 10%, or more, in high suburbs, this formula did not give acceptable results in all cases.

The McMath or St. Louis formula, a modification of the Burkli-Ziegler, was tried for a time. It gave better results, but did not prove satisfactory. The City then established 6 pluviometer stations for recording intensity, duration, and the amount of rainfall in the city limits and also installed automatic recording flow gauges in a number of sewers; and from the information thus collected, a new formula known as the "Philadelphia formula" was evolved. It was in reality an extension or modification of the Burkli-Ziegler and McMath formulas. This was an advance over previous methods of determining run-off, but did not give correct results under all conditions.

All these investigations led to the adoption of what is called "the rational method of designing sewers" which is based on data collected in Philadelphia and on facts evolved by the late Emil Kuichling, M. Am. Soc. C. E., in his work at Rochester, N. Y.

Finding that the streams into which sewers discharged were becoming very heavily polluted and that the maintenance of public health demanded that some corrective steps should be taken, the City of Philadelphia has adopted a comprehensive plan and has entered on the treatment of its sewage on a large scale. There is in operation a treatment plant on the Delaware River near the Torresdale intake to the City's water supply. At this plant, Imhoff tanks, sprinkling filters, chlorination, and sedimentation are successfully used.

In the northeast section of the city, a series of Imhoff tanks have been installed and are in operation, while in the southwestern section, preparations are under way for the erection of a large plant for sewage treatment.

MORRIS KNOWLES,* M. AM. SOC. C. E. (by letter).†—In discussing Mr. Fuller's paper, there is little fundamental information that can be added. The author covers the subject completely and concisely, and any addition must necessarily be an expansion of some item already mentioned.

The trends of the demand of people, as regards quality of water, is one factor which is seldom mentioned, but which has played an important part in the history of public water supply systems. As in everything else, the people are demanding an improved quality of water for their use. The time when it was necessary that a water be safe for drinking purposes only, has passed. Even during the past 10 to 20 years, the demand of the people as regards quality of water has grown. The people of New England dislike mud and hardness, but will put up with color. Western people have put up with turbidity and hardness, but dislike color. The time is not far distant when a water that has turbidity, color, or hardness will not be tolerated anywhere. In other words, water must not only be safe for drinking purposes, but must be pleasing in appearance and pleasant and easy to use.

It has been known for a long time that one of the prime requisites of an industrial community is soft water. This is being more and more realized and, consequently, the demand for such water is increasing. The importance of this factor is well shown by the number of softening plants that are being installed. In the State of Pennsylvania, there are twelve such plants, four of which supply a population of more than 50 000.

Another quality of water that is becoming important is its temperature. This is indicated by the fact that many plants equipped with condensers of the water-works type have had to discontinue their use, due to the fact that the few degrees rise in temperature of the water that results has been the cause of many complaints.

The demand for improved quality of water has been almost in direct ratio with the education of the people on the subject of water supply. As soon as they know that they can get a good quality of water, they will demand it. Although the people are better informed on the question of water supply than they have been in the past, there is still room for more education. In every line of endeavor, be it industry, art, or other activity, it is necessary that education of the people proceed with the progress that is made, or the progress will soon cease.

A great number of people, knowing nothing about water, revolt against drinking purified water that has come originally from a known rather polluted stream. In order to overcome that difficulty, either the pollution will have to be removed, or the people will have to be educated to the idea that the water is satisfactory in spite of its antecedents. In almost any city in the country where good water may be obtained by turning a tap, there is also a company

* Pres. and Chf. Engr., Morris Knowles, Inc., Pittsburgh, Pa.
† Received by the Secretary, November 4, 1927.

engaged in the business of selling so-called "spring", well or distilled water for drinking purposes. This may be an indication that the public water supply is not what it should be, but more often it is simply lack of education on the part of the people relative to the subject of water supply and safety of the water from public systems, as compared to the doubtful quality of other water.

The water-works engineer now has many problems dealing with the art and details of public water supply and the education of the people. He will probably always have these, but he has, and will have, assistance that will tend to make the solution of the problems less difficult. This aid is, and will be, given through the agencies of supervision, co-operation, and consolidation.

The developments during recent years in the fields of governmental supervision and inspection of drainage areas, of interstate agreements, and of consolidation of public water supplies, are of particular interest to, and should be welcomed by, the water-works engineer. It is in this direction that people must look for the solution of problems that have to do with the increasing pollution of streams by such material as industrial wastes and mine drainage that cannot be dealt with by present water purification methods. In order to solve these problems, engineers must either correct conditions at the source of the pollution, or go to one of the few remaining unpolluted streams for water.

Since about 1890, there has been a gradual change in the United States in conditions as regards sources of public water supply. Previous to that time, practically all American streams were relatively unpolluted, and the problem of locating a surface source for public water supply purposes consisted of locating a convenient stream having the necessary flow. With the greater stream pollution of the present day and the greater demand for sources of public water supply, the problem has become more complicated. In order to find a source of supply, a city, in many cases, must go outside its own jurisdiction and must look to State and even Federal administrative bodies for assistance.

State assistance and supervision of public water supply has become available through Sanitary Divisions of State Health Departments. In 1890, Massachusetts had about the only effective State Health Department in this respect in the country. Since that time practically all States have provided for Sanitary Bureaus and in more than one-half the States it is necessary to obtain the approval of these agencies on water supply and sewerage projects. People must look largely to these governmental agencies for the preservation of present sources of public water supply.

In the cases of large sources of water supply, Sanitary Divisions of State Health Departments are not capable of handling the question, as most large streams flow from one State to another. In such instances they must look to interstate agreements and sometimes Federal supervision, such as the U. S. Public Health Service. Interstate agreements are usually the result of meetings in which the interested State Governments, manufacturers, and others concerned are represented. The Interstate Stream Conservation Agreement concerning the Ohio River, in which Pennsylvania, Ohio, West Virginia,

Maryland, New York, Indiana, Illinois, Kentucky, and Tennessee are represented, is one example of the effectiveness of such procedure. The success of this co-operative group will probably lead to the development of other similar agencies and agreements, in an effort to solve other similar problems.

In solving the question of mine drainage pollution in Pennsylvania, the ultimate answer will probably be the preservation of certain drainage areas against such pollution for use as sources of public water supply. This is about the only satisfactory solution in sight and the Court decision in the Indian Creek case would indicate that this procedure is feasible. Such solution will involve not only activity and increased power on the part of the State Government, but also the probable formation of regional and other combined water supply districts, inasmuch as it would be impossible for the smaller cities to have their own protected drainage areas.

The development of the art and of the details of public water supply has been extensive and should continue, but one of the more important developments as regards public water supply will probably be in the field of co-operation and consolidation.

R. A. HART,* M. AM. SOC. C. E. (by letter).†—The paper by Mr. McCrory‡ leaves little to be desired. It is not made exactly clear that the work of the late Charles G. Elliott, M. Am. Soc. C. E., as Chief of Drainage Investigations, extended throughout the United States, and that his staff was concerned with the drainage problems of the humid as well as the arid regions.

With characteristic modesty, the author has refrained from mentioning his own contribution to the history of land drainage in the United States, but it is a well-known fact that he and his staff have played a dominant part in establishing land drainage on a scientific basis and have been directly instrumental in forwarding large drainage developments.

With respect to drainage of irrigated lands, one additional thought is worthy of consideration. Such drainage has to do with colonized, improved lands, in which large investments have been made, and which represents redemption of lands formerly highly productive, or the protection of lands now highly productive, rather than the reclamation of lands of relatively low market value and productiveness, as is the case with humid lands susceptible of drainage. The redemption or protection of irrigated lands by drainage often is the sole method of saving the original investment.

L. L. HIDINGER,§ M. AM. SOC. C. E. (by letter).‖—Mr. McCrory has assembled interesting and fairly complete data on the most important phases of the subject. However, it occurs to the writer that the section, "Construction of Open Ditches"¶ might have covered a somewhat wider field.

Mr. McCrory refers to the fact that floating dipper dredges have been largely replaced by drag-line excavators. This change has made the con-

* Secy-Mgr., Western Clay Products Assoc., Salt Lake City, Utah.
† Received by the Secretary, November 4, 1927.
‡ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1628.
§ Pres., Morgan Eng. Co., Memphis, Tenn.
‖ Received by the Secretary, November 7, 1927.
¶ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1630.

struction of more stable side slopes possible. Prior to 1920 practically all the dredging for open drainage channels was accomplished by floating dipper dredges. This type of dredge is a very unwieldy machine. It is impracticable to dig side slopes flatter then 1 to 1 with this type of equipment and, as a rule, the slopes are left considerably steeper. Some of the first work was laid out with side slopes of 1 on ½, because it is not economically practicable to dig flatter slopes with a floating dipper dredge. This was done with the expectation that the banks would weather down to a flatter slope and still leave sufficient capacity in the drainage channel. The floating dredge was almost invariably too large to dig short laterals to a properly designed cross-section, so that deterioration from the side slopes was not considered a serious defect, provided they were laid out deep enough to allow some sedimentation in the bottom without impairing the drainage of adjacent lands.

The substitution of drag-line excavating equipment for floating dredges has made possible the construction of drainage channels with any side slope that may be considered stable. Since the common use of this type of equipment, side slopes have been changed from 1 on ½ to 1 on 1½, or flatter, as may be deemed necessary. The present tendency is to dredge open drainage channels with 1 on 2 side slopes where they are located in ordinary alluvial soils, and flatter where the material is unstable.

Another important change in the construction of open drainage channels that has come about through the use of drag-line equipment, is a better balanced drainage system. When floating equipment was in common use it was cheaper to excavate small laterals with the dredge than by other means. This resulted in over-sized laterals that left an unbalanced drainage system. It is a recognized fact, among well informed reclamation engineers, that it is not economically feasible to construct channels for land drainage purposes large enough to carry off rainfall as fast as it is precipitated, and that part of it must be stored temporarily. Such being the case, it is imperative that a drainage system be balanced as carefully as possible so that lateral drains will not have excess capacity and thus overload the outlet channels. The drag-line has made possible the construction of drainage channels in accordance with the design, so that a better balanced drainage system can be installed and the flooding of lower lying lands can be largely prevented.

TABLE 6.—Capacity of Drainage Systems.

Location.	Year.	Cubic feet per second per square mile.
Southeast Missouri	1910	7.4
Northeast Arkansas	1912	8.8
Southeast Arkansas	1913	13.5
Southeast Missouri	1924	15.4
Northeast Arkansas	1925	17.4
Southwest Arkansas	1925	20.0

There has been a steady increase in the designed capacity of drainage channels. The first run-off curves provided only about one-third the capacity

that is now used. Table 6 shows the increase in capacity of drainage systems located in the Lower Mississippi River Valley within the last 18 years. It is based on an area of 300 sq. miles.

ROY N. TOWL,* M. AM. SOC. C. E. (by letter).†—It has been the writer's privilege during recent years to study silting problems affecting the maintenance of drainage ditches in the Missouri River Valley from Yankton, S. Dak. to St. Louis, Mo. The average fall of the river is about 0.8 ft. per mile. The banks are several feet higher than the sag of the valley between the river and the bluffs, which latter rise to an elevation of 200 to 300 ft. The natural drainage of the valley is generally parallel with the river for a distance of 10 to 20 miles before finding a natural outlet or making an artificial ditch outlet. Hence, it is necessary to receive small tributaries directly into the drainage system and, consequently, many drainage systems have been destroyed with silt deposits or seriously burdened, requiring frequent and expensive clean-out operations.

About twenty years ago, W. E. Pratt, County Surveyor at Tekamah, Nebr., protested to the County Board against allowing Silver Creek, with a water-shed of 25 sq. miles, consisting of loess bluff formation, to enter the main drainage system without first spreading over its delta and partly de-silting its flood waters. Also, in the same water-shed, York Creek having a like drainage area, filled the main drainage ditch with silt to about 65% of its carrying capacity in one flood. In both these cases, the silt-laden flood waters have been diverted from the drainage system and carried directly to the Missouri River across the natural line of drainage. The drainage ditches are carried through undersluice-ways at crossing points. The floodway for the silt-laden stream must have a silt-transporting velocity. It is also found desirable to provide a spillway at or near the point of diversion to relieve the floodway of excess waters which may be clarified in a relatively small area before reaching the drainage system. Such diversion ditches and settling basins, separately or combined, have been used by a number of drainage districts in Iowa, Nebraska, and Missouri since 1915, with very satisfactory results.

In most cases artificial delta formation is limited to a small area for a period of 6 to 10 years and then transferred to another limited area, etc. It is desirable to de-silt flood waters on a natural or artificial flood-plain. This may be readily accomplished by removing levee embankments on one or both sides of the offending tributary stream and permitting the overflow to be distributed at a shallow depth for a considerable distance, and flooding the adjoining land, as in irrigation, and then providing for drainage of clarified waters into the main system. In certain cases of this kind, corn has been successfully raised without any loss from flooding and with a gradual deposit of rich soil on a well-drained slope on account of the inundation. This result may be expected if standing water is removed in a reasonable length of time.

Undoubtedly, this is the oldest chapter in drainage and flood regulation found in the book of Nature. The writer anticipates that it may be adapted

* Civ. Engr. (Towl, Nelson and Schwartz), Omaha, Nebr.
† Received by the Secretary, November 9, 1927.

successfully as a part of the program even for streams of the magnitude of the Colorado River. In support of this proposition he would suggest a study of the stabilized channel below Yuma, Ariz., which existed for possibly fifty years prior to attempted regulation.

Desirable channel conditions, especially with respect to maintaining drainage ditches, have been and always will be subjects for careful consideration. In this connection the writer has observed many natural and artificial drainage outlets subject to submergence and back-water from the Missouri River, which apparently were adverse; but actually the hydraulic flood grade line indicated better flowage characteristics than would seem possible. In fact, submerged silt deposits are carried out in flood time and the coefficient of roughness is very much less than that found in upper portions of the same channel not subject to back-water, but which are invaded with growth of vegetation.

Increased low-water flow in the Illinois River at Beardstown, Ill., which maintains a wider and smoother channel than would otherwise be possible, has a beneficial effect. The increased discharge at low stages has been approximately equivalent to the increased flowage from Lake Michigan during the past 20 years without any increase in stage for said increment of inflow. Thus, it may be expected that, in connection with detention and storage reservoirs, better channels will result on account of improved low-water cross-sections. The writer is also studying the problem of slack low-water flow in its relation to maintaining wider and better cross-sections in drainage channels. In this case he has found many natural examples of channels with irregular bottom profiles and fairly good, uniform flood profiles.

In conclusion, the writer must admit that his best lessons are exemplified in natural streams and more forcibly in artificial construction that refuses to become stabilized in conformity with the adopted plans and specifications for the project.

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INDUSTRIAL WASTE DISPOSAL

A SYMPOSIUM

Discussion*

By Messrs. George W. Fuller and Almon L. Fales.

George W. Fuller,† M. Am. Soc. C. E.—Mr. Crichton's paper deals with an important and many sided problem. He has presented well its several phases which involve conflicting interests of unusual magnitude and numerous technical details. The Sanderson case‡ dealt with the matter individually, while in the Indian Creek case§ the decision is founded on the interests of the public. The Sanderson case, which has meant so much for forty years in Pennsylvania, was never accepted by the Courts of last resort in any other State, except Indiana, the decisions in other States being less favorable to nuisances created by basic industries.

Mr. Crichton has stated that the coal mining companies are now building a tunnel to discharge their mine drainage into Indian Creek below the impounding dam from which the Indian Creek water supply of the Pennsylvania Railroad and subsidiary companies is obtained. This means that the pollution problem will simply be transferred to the lower reaches of the stream. This is an expedient that has been followed before in the case of some of the reservoirs of Allegheny County, at Johnstown, and in the Altoona District. Such diversions, aside from postponing the ultimate solution of the pollution problem, introduce another problem as to the capacity of existing reservoirs. More waters must be stored to compensate for the diversion. The speaker has too great an appreciation of the difficulties involved in connection with mine drainage to believe that a simple permanent solution may be found along the line of legislation alone. They must be dealt with in a practical way along the line of a comprehensive program that will be fair to conflicting

* This discussion (of the Symposium on Industrial Waste Disposal presented at the meeting of the Sanitary Engineering Division, Philadelphia, Pa., October 7, 1927, and published in September, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr. (Fuller & McClintock), New York, N. Y.

‡ *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1657.

§ *Loc. cit.*, p. 1659.

interests. Such a program is contemplated in the creation of the Sanitary Board, which has set up a classification of the different kinds of water-sheds in Pennsylvania.

In this connection the speaker recalls the handling of some large projects in Great Britain, particularly the development of a large water supply in the midland country in England north of the City of Sheffield, involving cities like Nottingham, Leicester, and others. In Great Britain the rights of lower riparian owners are satisfied by compensation in kind, which ordinarily is taken to mean the release of water at times of dry weather roughly equal to the ordinary dry-weather flow of the stream. In the Sheffield instance, it is said that little or no impounded water was released at times of extreme drought as a supply of water can be taken from the lower reaches of the river and softened, filtered, and delivered to the vicinity of the impounding reservoir for use of riparian owners below. The speaker does not suggest, of course, this particular arrangement for the project under discussion, but merely cites it as an example of the way in which, from the standpoint of public policy, attempts have been made to balance equities and conveniences elsewhere. Surely there will come some way by which sizable communities may procure satisfactory water supplies and yet let industries flourish.

ALMON L. FALES,* M. AM. SOC. C. E. (by letter).†—This paper shows that very valuable studies on tannery wastes disposal have been made during the past two years under the direction of Messrs. Howalt and Cavett for the Tannery Waste Disposal Committee of Pennsylvania. As the authors have pointed out, their investigations thus far have been confined to the so-called partial or preliminary treatments. They have studied the effects of discharging the continuous-flow wastes without treatment, and mixing together the more objectionable intermittent wastes followed by uniform discharge into the stream during eight hours without sedimentation, after sedimentation alone, and after sedimentation followed by chemical precipitation, the continuous-flow wastes being included in the last-named treatment. Investigations remain to be made of the sludge disposal problem and of methods for the complete treatment of the wastes.

In considering the effects of the discharge into a stream, of untreated wastes or of mixed wastes, including sludge produced by the interaction of different wastes, it must be borne in mind that organic solids, deposited in the bed of the stream, tend to rob the overlying water of the dissolved oxygen required to maintain the stream in an inoffensive condition. Organic matter deposited in a stream in cold weather, when there is little or no bacterial activity, unless flushed out by high stream flows, will undergo decomposition during the following summer, thus adding to the current oxygen demand load on the stream. The effects of such deposits may have influenced the results of the studies during the second summer when the settleable solids were being removed from the wastes.

A period of 8 hours each day for uniform discharge of wastes or effluents was presumably selected because the bulk of the wastes are produced during

* Cons. Engr. (Metcalf & Eddy), Boston, Mass.
† Received by the Secretary, October 27, 1927.

this period, and the operation could be confined to one shift. From the point of view of the condition of the stream, it is desirable to discharge the wastes uniformly with respect to the stream flow. If the stream flow is substantially the same during the 24 hours, it is desirable to distribute the load as uniformly as practicable throughout the day and night.

It is stated* that the tannery selected for the studies is representative of the sole-leather tanneries operating with the vegetable tanning process in general use. Variations were found in the composition and strength of various wastes at different times during the day and on different days. Still greater differences may occur between the wastes from different tanneries. In view of the differences in the quality and quantity of wastes, variations in the character and volume of stream flow available for dilution, and other varying local conditions, an investigation should be made at each tannery to determine the extent and kind of treatment required to satisfy the local conditions. Studies such as those made by the authors are of great advantage in solving these problems.

It may be helpful to describe briefly some of the tannery wastes treatment investigations and experience of the writer and of the firm of which he is a member.

The writer was first introduced to tannery wastes more than twenty-five years ago at Worcester, Mass., where the wastes from a large sole-leather and belting tannery are discharged into the city sewer system. The tannery wastes in combination with the acid-iron wastes from the wire mills and foundries caused the sewage to be colored black, blue, or brown, much of the time, and the color was not always entirely removed by the method of chemical precipitation with lime, then in use. In this connection, it should be emphasized that if sulfate of iron, or copperas, and lime are used for chemical precipitation of tannery wastes, it is important to maintain an excess of lime in the wastes to prevent the formation of tannate of iron, which is an ink. It is reported that in recent years the quantity of lime in the wastes discharged by the tannery at Worcester has increased, causing precipitation of large quantities of hydroxide of iron in the new Imhoff tanks, and greatly increasing the quantity of sludge.

At Gloversville, N. Y., the tannery wastes constitute such a large proportion of the sewage of the city, that for studying the sewage disposal problem, a testing station was run for about a year in 1908 and 1909 to determine how the tannery wastes would affect the different methods of sewage treatment. It was found that these wastes more than doubled the strength of the sewage, but did not render the methods of sewage treatment inapplicable. A very good effluent was produced by preliminary sedimentation, trickling filter treatment, secondary sedimentation, and intermittent sand filtration, and a treatment plant was built embodying these features. This plant has functioned well, but difficulties have been experienced in handling the sludge because of excessive quantities of lime, hair, and fleshings in the tannery wastes, which were expected to be removed by tanks at the tanneries prior to discharge into the sewers.

* *Proceedings*, Am. Soc. C. E., September, 1927, Papers and Discussions, p. 1677.

About 1900, a question arose as to whether the wastes from a tannery at Ballston Spa, N. Y., would be susceptible of treatment by bacterial methods. It was found that although certain individual wastes from the tannery were sterile, the combined wastes contained considerable numbers of bacteria which increased on incubation.

At the C. S. Harriman Tannery, in North Wilmington, Mass., where heavy leathers are manufactured, studies made in 1913 indicated that the combined wastes could be treated satisfactorily by sedimentation and intermittent sand filtration, and a plant built for such treatment has proved very satisfactory. The effluent is discharged into a small brook. The sludge formed in the sedimentation tanks is removed periodically and de-watered on small sand beds. The de-watered sludge is reported to be excellent material for application to grass land. The sludge, when pumped to the sludge beds, created a strong hydrogen sulfide odor which was very objectionable for several days. This trouble has been overcome by applying creolin or crude coal-tar disinfectant to the sedimentation tank the night before cleaning, after the tank has been put out of service.

The wastes from the tannery of the White-Son Company, in Walpole, Mass., manufacturers of fancy leathers, are quite different from the wastes produced by the heavy leather tannery in North Wilmington. Nevertheless, similar treatment by sedimentation and intermittent sand filtration has been satisfactory. The effluent is discharged into a small mill-pond. The sludge is drawn about once a month to one of the sand filter beds and the de-watered sludge is used for filling low land near-by. This sludge, which is largely spent ground sumac, has no value, but is unobjectionable in character.

At the tannery of the J. G. Curtis Leather Company, in Ludlow, Pa., a detailed investigation of the wastes disposal problem in 1916 and 1917 indicated that the most practical method of treatment was to bring the different wastes together for interaction and sedimentation, and a plant for this treatment was recommended, including pumping station, sedimentation tanks, and sludge beds, with provision for the construction of works for more complete treatment if and when such treatment should become necessary.

At the Buckman Tanning Company Factory, in Woburn, Mass., where heavy leathers are made and subsequently split for the manufacture of patent leather, earth basins were originally provided for the wastes, but without adequate means of sludge removal. The effluent from this plant seriously polluted the brook into which it was discharged and the plant itself created a serious odor nuisance. The hydrogen-sulfide gas given off turned all the white houses in the neighborhood black, due to the formation of lead sulfide. The sedimentation basins were rebuilt and provision was made for removal of the sludge, thus greatly reducing the odors. It was found that by treatment of the wastes with sulfate of alumina solution, using sulfuric acid for the control of the hydrogen-ion concentration, or p H value, a well clarified effluent could be obtained; and this treatment, although not entirely satisfactory, is being used pending the construction of a sewer which will connect the tannery with the Metropolitan Sewer System. One difficulty experienced in this treatment is that the reduction in alkalinity causes hydrogen-sulfide

to be released to a greater extent on agitation of the wastes, but the odor from this source is much less than that formerly produced by the earth basins.

At the tannery of Winslow Brothers and Smith Company, in Norwood, Mass., where sheep, goat, and calf skins are tanned, and wool is scoured, it was found advisable to separate the wastes into three classes: Rinse waters from the wool department; wool scouring wastes; and combined wastes from the tannery and wool scouring wastes degreasing plant. The effluents are discharged into a small brook emptying into the Neponset River, which is already polluted by paper mill wastes.

The rinse waters from the rinsing of wool after scouring, contain bits of wool, but comparatively little other organic matter. They are passed through plate screens having $\frac{3}{32}$-in. perforations and then discharged directly into the stream. The wool removed by the screens is returned to the wool department.

The wool scouring wastes are exceedingly turbid and strongly alkaline. They contain much suspended and dissolved organic matter including considerable grease—sometimes as much as 2 or 3 per cent. These wastes are discharged into tanks for sedimentation and cooling, the resulting sludge being pumped from time to time to the sludge beds. The settled wastes are pumped to "acid-cracking" tanks where they are acidified with sulfuric acid to precipitate the grease. The grease separates partly as sludge at the bottom and partly as scum at the top, and the clarified "middle liquor" is drawn off to the channel carrying the wastes from the tannery. The grease sludge is drawn off to "magma" beds of sand, and, after draining and drying, is wrapped up in pieces of burlap to form "puddings" which are placed on metal plates in the steam boxes of hydraulic presses where the grease is melted and pressed out, leaving a fairly dry, thin cake which has been used to some extent for fertilizer by farmers in the vicinity. The expressed oil is separated from the water by flotation in tanks and refined by boiling with sulfuric acid, forming "acid wool degras" which is put on the market.

The combined wastes from the tannery and degreasing plant are pumped to sedimentation tanks. The resulting sludge is drawn to sludge beds every 2 to 4 weeks in warm weather and every 3 to 6 weeks in cold weather. The de-watered sludge is used in part for fertilizer by farmers in the vicinty and the remainder is hauled to waste land.

During periods of cold weather and high stream flow, the sedimentation tank effluent is discharged into the stream without further treatment. Sedimentation accomplishes the removal of about 50% of the suspended solids, but the effluent is still colored and extremely turbid, and contains much finely divided suspended and colloidal organic matter.

During periods of warm weather and low stream flow, the settled wastes are treated by chemical precipitation with sulfate of alumina, the hydrogen-ion concentration, or p H value, of the wastes being controlled by the addition of sulfuric acid to give the most efficient and most economical treatment. The wastes are usually very strongly alkaline. Whenever there is a deficiency in alkalinity, milk-of-lime is added to the wastes. The combined wastes vary greatly in strength and composition during the day, and it is necessary to

equalize the quality of the wastes, as by passage through the sedimentation tanks, prior to chemical precipitation. The chemically treated wastes are passed through tanks and the resulting sludge is discharged periodically on to the sludge beds.

The chemical effluent is well clarified, but contains considerable dissolved organic matter and is quite putrescible. This effluent is further treated by intermittent filtration to the capacity of the available area of sand filter beds, and the remainder is treated by dilution in the brook with water drawn from a reservoir owned by the Company. During periods of extremely low flow, practically all the tank effluent is filtered. The filter-bed effluent is practically clear and colorless and non-putrescible.

Experimental investigations of the treatment of the settled wastes by the trickling filter and activated sludge processes were carried out on a working scale, and it was found that a satisfactory effluent could be produced by either process, if all the wool scouring wastes were degreased prior to discharge into the tannery wastes channel. It was found that the activated sludge process would entail a considerably smaller capital expenditure, but much larger annual expense for operation and maintenance. An experimental sand filter bed was operated in conjunction with the experimental trickling filter, and it was found that the trickling filter effluent could be applied to the sand bed at a high rate and produce a highly purified effluent. By following the method of treatment previously described, it has not been necessary to build either trickling filters or an activated sludge plant.

It has been pointed out that local conditions will govern the extent of treatment required at different places. It is also true that the extent of treatment required at a given place will vary with varying conditions of temperature and stream flow at different seasons of the year. This fact is well illustrated by the regulation of the treatment of the wastes from the Winslow Brothers and Smith Company tannery according to the requirements of the stream.

To recapitulate, during the winter, when there is little or no bacterial activity in the stream, and, in the spring, when the stream flow is relatively high, only the strongest of the wool scouring wastes are degreased and the other wastes are treated only by mixing and sedimentation. As summer approaches and the stream flow decreases, all the wool scouring wastes are conducted to the degreasing plant and water is drawn from the water supply reservoir for dilution of the sedimentation tank effluent. If the water-shed and reservoir were large enough, no further treatment, of course, would be required; but it is necessary to conserve the water and provide a more complete treatment of the wastes. The next step is the starting of chemical precipitation of the sedimentation tank effluent and regulation of flow of reservoir water drawn for dilution. Subsequently, the filter beds are put in operation.

By such regulation of the treatment, it is possible with a minimum expenditure, to produce an effluent at all times, which will meet the stream requirements.

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PAPERS AND DISCUSSIONS

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NEW THEORY FOR THE CENTRIFUGAL PUMP

Discussion*

By Messrs. Max M. Munk, Elmo G. Harris, and O. Gruenberger.

Max M. Munk,† Esq. (by letter).‡—The writer would like to register his disagreement with the author's conclusions, believing the old established blower theory to be correct.

In declaring the pressure to be "part and parcel of the velocity",§ Professor Sherzer flatly denies to the fluid pressure an existence of its own. This is in contradiction to the most fundamental conceptions. Each fluid particle is under the definite action of the adjacent particles, regardless of its motion together with them. The intensity of this action is called the pressure. How definite it is is demonstrated by the fact that the density of the fluid is influenced by the pressure. Velocity and pressure do exist independently of each other. Even the untrained man understands that perfectly well if he has ever been in a dense crowd. He moved with the crowd at a certain speed, and independent therefrom was pressed more or less by his neighbors.

The pressure created by the centrifugal force is not even a function of the exit velocity, U, only. The rotating fluid does not necessarily extend to the center, and the pressure created depends on the width of the rotating fluid ring. If the entrance radius is almost as large as the exit radius, the fluid hardly receives any increase of pressure, although it may be impelled to the same exit velocity, U, as a fluid entering at the center. The rotation of this narrow fluid ring, rather than the case of a rotating fluid disk, is illustrated by the whirling hammer.

As a matter of fact, good centrifugal pumps may furnish a pressure surpassing one-half the theoretical maximum pressure, that is, surpassing the pressure claimed as maximum by Professor Sherzer.‖

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* Discussion of the paper by A. F. Sherzer, Assoc. M. Am. Soc. C. E., continued from December, 1927, *Proceedings*.

† Research Dept., Westinghouse Elec. & Mfg. Co., East Pittsburgh, Pa.

‡ Received by the Secretary, November 15, 1927.

§ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1777.

‖ Compare, for instance, "Die Kreiselpumpen," by C. Pfleiderer, Berlin, 1924, p. 97.

ELMO G. HARRIS,* M. AM. SOC. C. E. (by letter).†—It is evidence of a certain kind of courage on the part of the author to announce that all the heretofore recognized present-day authorities on centrifugal pumps are entirely wrong.‡ That is a broad statement. The writer cannot concede that the author has established any one of the three claims made in the Synopsis.§ They are so sweeping that to disprove all of them systematically would require a treatise on the subject. It would be inappropriate to fill these pages with matter that can be found in many recent publications. However, the following comments may be appropriate.

(*a*).—The quotation from John Perry§ and the author's statements‡ will apply to many of the writings that are more than twenty-five years old, but not to the more recent ones.

(*b*).—The author is right in declaring that Bernoulli's theorem should not be applied in the case of water flowing through a centrifugal pump, for energy (head) is being continuously added to the water as it passes through the pump. Moreover, the pressure is not constant over any cross-section of a passage between propellers. It is relatively high next to the front side of a propeller and low on the back side. Doubtless, in some defective designs, a vacuum occurs at the back side of the runners. Also, in attempting to apply Bernoulli's theorem, some writers have used relative and absolute velocities indiscriminately to the utter confusion of the subject.

(*c*).—Under "Current Fallacies",‖ the author's analogy of the "hammer" is incomplete. He should have considered a smooth rod with hammers strung on it from the center of rotation out to the last one.

(*d*).—The writer thinks that the author is entirely wrong in his statement¶ that the head in the water at "shut-off" is $\frac{U_a^2}{2g}$ instead of $\frac{U_2^a}{g}$. This pressure head at shut-off has been the cause of much confusion, especially among the earlier students. It led one writer to state (in a book on centrifugal pumps) that the curve of the vanes was of no consequence. The confusion results in studying, and deriving conclusions from experiments on, pumps that are not in operation—not discharging, although revolving.

The author states‖ that "there is a pressure created by centrifugal force which is equal to $\frac{U_a^2}{2g}$ and which is part and parcel of the velocity itself. The total head may be regarded as either, but not both". This statement does not make anything clear nor remove confusion. By the laws of mechanics the head, $\frac{U_a^2}{g}$, is there, although it cannot be utilized until the pump is discharging.

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† Received by the Secretary, November 21, 1927.
‡ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1776.
§ *Loc. cit.*, p. 1775.
‖ *Loc. cit.*, p. 1777.
¶ *Loc. cit.*, p. 1778.

(*e*).—The "Derivation by Principle of Angular Momentum" is correct, although given by the author as incorrect.* Equation (*C*)† can be easily transformed into the writer's Equation (37),‡ discussed later. However, the writer prefers to use plain reaction of a jet to the change in angular momentum. The later term does not look familiar to ordinary students of hydraulics.

(*f*).—Under "Reasons for Observed Discrepancies",† no adequate reasons are given.

(*g*).—As bearing on most of the matter relative to "Real Action of Centrifugal Pump",§ in the references cited later, it is proved that with β_2 fixed, it is theoretically immaterial whether the water moves out under purely centrifugal action producing pressure; or centrifugal action producing acceleration; and it is immaterial as to the point between center and periphery at which the water is picked up; and that the same effect will be obtained by short vanes or cups picking the water up impulsively; and that the writer's Equation (37) previously referred to, applies either with or without back pressure at the discharge. The discovery of this fact was a surprise and a revelation to the writer, and it much simplifies the whole subject. For these reasons the general term, centrifugal pump, is a misnomer. Engineers may be more ready to accept these statements if they will consider the action between a golf club and the ball. Also, consider the theory (nearly realized) in the Pelton water-wheel. By the laws of mechanics, if the rotation of the wheel is reversed and water is fed to the cups, it will be thrown off with a velocity twice that of the cups. Also, consider reversing the rotation of an axial flow turbine.

(*h*).—Under "Experimental Verifications"‖ the author interprets results by his own erroneous theories. He seems not to have given due recognition to the fact that for small discharges (as compared with discharge at best efficiency) most of the work put into the pump is dissipated. Moreover, the observations under experiment cannot be properly interpreted without knowing the area of the discharge outlet, of the pump, and the angle, β_2. With this information and by the use of the writer's Equation (37) discussed subsequently, the experiments could be correctly interpreted—not otherwise.

(*i*).—Under "Effects of Casing":¶ If, in the experiment, the pressure head, h_2, at the line of discharge (Point 2, Fig. 1)** had been determined, the total head in the water at the discharge line would be $\frac{C_2^2}{2g} + h_2$, and if this were plotted on Fig. 6,¶ the line would lie above the "Actual Head" line—the difference being the loss of head beyond the discharge line.

In the analogy of the paddle in a tank¶—whence comes the jet which, by inference, has a higher velocity than the paddles?

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1778.
† *Loc. cit.*, p. 1779.
‡ "Compressed Air," 1917.
§ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1781.
‖ *Loc. cit.*, p. 1782.
¶ *Loc. cit.*, p. 1787.
** *Loc. cit.*, p. 1779.

(*j*).—Under "Pump Considered as Three Orifices",* the author is not correct in considering as orifices the three sections that he selects in Fig. 8.† The sections, 2 and 3, are not orifices. He is also incorrect in computing the drop in pressure head at each so-called orifice as that which would occur if the water, at each place, started from a state of rest. His calculations‡ show only slight differences of velocity at the three sections and what change occurs will be gradual. The three-orifice theory is groundless.

Under "Applications of New Theory":§ The outstanding item of interest in the paper lies in the fact that the author has designed a very efficient pump, but that is of little value unless he will give all its dimensions, angles, etc. With these data, the results of the experiment can be interpreted in the light of other theories.

The writer is unable to see that the three-orifice theory influenced the design in any way. The author states, in reference to the "Results of First Design",‖ that "the runner, of the double-suction type cast in bronze, was of ordinary design and workmanship".

When, if at all, the dimensions and angles of these pumps are revealed, there will be shown a small angle between the backward tangent and V_a, Fig. 1 ($180 - \beta_2$), and, when computed, V_a will be found nearly equal to U_a. This will make the absolute velocity, C_2, nearly radial and relatively small. (It is not possible to make it radial.) Thus, there will be no "drag" between the water in the circular receiving chamber and that escaping from the runners; and there will be but little kinetic energy to lose. If the author will put in a runner with the same forward pitch that it now has backward pitch (β_2 between 20° and 30°), the efficiency will drop below 50 per cent. This drop will be caused by the loss of most of the kinetic energy in the water, due to the drag against the relatively slow moving water in the outside circular receiving chamber. If the author will attach a pressure gauge at the discharge line (Point 2, Fig. 1), he will find most of the final head registered there as pressure.

This pressure will approximate $\frac{U_a^2}{2g}$ (although always less) when the vanes are turned backward as specified. However, when the rotational velocity of the water at Point 2 is computed ($= C_2 \cos \alpha_2$), it will be found to be relatively small, altogether too small to develop the pressure by centrifugal force, hence, the writer's statement that the term, centrifugal pump, is a misnomer when applied to the majority of such pumps as are now made. Fortunately, the writer's Equation (37), developed subsequently, covers every case with the qualification that it does not include dissipated energy.

In reference to guide-vanes: They should be used with caution, if used at all. They can function properly only with one combination of velocity of rotation, quantity of discharge, and angles such that the relative motion of

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1789.
† *Loc. cit.*, p. 1791.
‡ *Loc. cit.*, p. 1792.
§ *Loc. cit.*, p. 1793.
‖ *Loc. cit.*, p. 1794.

the water will be tangent to the vanes at their tips. Since the combination of quantity and speed of rotation that will give best efficiency cannot be determined except by test after the pump is built, it may be advisable to change the angles of the guide-vanes slightly after testing the first pump built for a stock design.

Many years ago the subject of centrifugal pumps was young as compared to the present knowledge of the theory and to the present perfection of designs and excellence of performance. The writer sought, through the available literature, for something about centrifugal pumps, that he might impart the information to his class. He found the field blank so far as his search extended, except that in the accepted highest authority on civil engineering at that time—Trautwine's "Civil Engineer's Pocket-Book"—he found, in substance, the statement that the centrifugal pump was a low-efficiency machine in which the lift was practically limited to 27 ft.

Above this lift the efficiency was said to fall off rapidly. The writer, therefore, started to prepare a lecture for his class. The preparation of that lecture took up his otherwise spare hours for a year or two. The findings were published by the Society* in 1903, and, again, in somewhat better form in his textbook.† In both these publications can be found derivations of the writer's Equation (37):

$$H = \frac{U_a^2 + V_a U_a \cos \beta_a}{g}$$

Note that $\cos \beta_2$ is minus when β_2 exceeds 90 degrees. In this, the symbols are those used by the author and shown on Fig. 1, except the symbol, H, which is discussed later. This formula is derived by three independent lines of approach. It is derived by Daugherty‡ and also by Slocum.§ That it is based on correct assumptions as to the underlying mechanical principles seems beyond successful controversion. The writer prefers his Equation (37) to any other that he has found. The angle, β_2, is a constant for any one pump. It is under the control of the designer. The speed of rotation, U_a, is under the direct control of the operator. V_a can be computed directly when the quantity of discharge has been measured. Therefore, it is evident that the H of the writer's Equation (37) can be quite accurately computed in any well-controlled experiment. It is the control of the chief characteristics of the pump by the angle, β_2, that gives the formula value in designing. It is in the interpretation of H in the formula that the experimenter and the designer need to use caution and judgment.

Let Q be the weight of water moved per second. If no energy is dissipated between intake and discharge of the pump, the work required to drive the pump would be QH; but there are always losses due to several causes. These cannot be measured or computed separately, but the total can be determined and separated into inside and outside losses.

* *Transactions*, Am. Soc. C. E., Vol. LI (1903), p. 166.

† "Compressed Air," 1917, Chapter VIII, "Fans".

‡ "Hydraulics," 1925, p. 311, Formula (169); also, "Centrifugal Pumps," 1925, p. 62, Formula (36).

§ "Elements of Hydraulics," 1917, p. 262.

Let D equal the dynamometer record of rate of work put into the pump, in foot-pounds; let F equal the dissipated energy between the intake and the discharge, in foot-pounds; and H_1, the head lost between the intake and the discharge. Then,

$$F = D - QH,$$

and

$$H_1 = \frac{F}{Q}$$

Let H_2 equal the head in the water immediately after escaping from the propellers at the line of discharge, referred to still outside water; let h_2 equal the pressure head measured by a gauge at the line of discharge, plus the height of the gauge above still outside water (suction lift); and C_2, the absolute velocity at the line of discharge. (To be computed after determining V_a.) Then,

$$H_2 = \frac{C_2^2}{2g} + h_2$$

If H_3 is the pressure head in the discharge pipe measured by a gauge as near the pump as possible, but clear of the eddying effect of the discharge from the pump, plus the height of the gauge above still water; and H_4 is the head lost due to imperfections in the fixed parts of the pump, made to receive the discharge and conduct it into the plain pipe; then,

$$H_4 = H_2 - H_3$$

So far as the writer knows, this means of locating the losses in a pump and finding their relative amounts has not been clearly shown before. It may give some light to designers who are trying to improve the efficiency of their pumps.

Other interesting, instructive, and valuable conclusions can be reached from the writer's Equation (37) as a starting point, but, since most of them appear in the papers cited, they will not be given here. However, the following historic notes will be interesting and will serve to show how the leading men in practice in this subject were floundering in ignorance twenty-five years ago.

About 1903 the writer tried to interest a pump manufacturing company (one of the largest) in some patents that he had taken out. After courteous attention and some days of consideration, the manager announced that, unknown to the public, the company had for several years conducted experiments to develop a centrifugal pump that it would be willing to advertise and sell. Having spent $300 000 in the effort, it was then about ready to put a pump on the market and it could not afford the delay and cost of starting on a new line.

About the same time and with the same object, the writer was discussing his Equation (37) with the chief engineer of another company that was making and selling centrifugal pumps. This man stated unqualifiedly that the highest head that could be attained by any centrifugal pump, regardless of the shape of the propellers, would be $\frac{V_a^2}{2g}$, and as conclusive proof he cited

that his company had tested several pumps, and in every case the highest head was observed at "shut-off", and that head was, for every pump, $\frac{V_a^2}{2g}$. Any one who is able to interpret the writer's Equation (37) can find the fallacy of this argument and conclusions.

In 1894, there appeared a book entitled "Centrifugal Pumps". It was written by an American manufacturer and was probably accepted as authoritative. In this book appears, in substance, the statement that water must be held in the pump through at least one revolution in order to give it the necessary "send", and that the advantage in turning the vanes backward (β_2 greater than 90°) is that the water is given a final shove outward.

While the derivations are given in the references cited, the one that the writer prefers is given here for the convenience of the reader. It is direct, short, and clear. Referring to the author's Fig. 1, it may be assumed that the pump takes up the water from a state of rest; and, to get the basic mechanical principles involved, all dissipation of energy will be neglected. This dissipation consists of surface friction, friction of water on water (eddies), and, in some poor designs, spraying of water through a vacuum. It may be further assumed that the revolving wheel is immersed in water under pressure (this pressure may be either greater or less than that of the atmosphere).

The static pressure of the water on the outer surfaces of the wheel is everywhere perpendicular to the surface (this surface includes the peripheral area of discharge of width, b_2). Evidently, this perpendicular pressure neither aids nor retards rotation. When, and where, the water is picked up by the runners it exerts a dynamic pressure (reaction) resisting the imparting of velocity. Thus, the runner imparts energy to the water. This energy (pressure and motion) is retained by the water and more is added as it passes outward under the influence of the propellers. However complex this action is at the pick-up (Point 1, Fig. 1), and between there and the point of discharge (Point 2, Fig. 1), it is all a part of work done by the pump in preparation (one might say) for its final discharge, at Point 2, against the back pressure and with a velocity, C_2. Since static pressure in the outside water does not resist rotation, the resistance of the pump to rotation must be that due to reaction of the water escaping at the periphery.

If Q is the weight of water escaping in 1 sec., the reaction in the reverse direction of C_2 is $\frac{Q\,C_2}{g}$, and the component opposed to tangential motion is,

$$\frac{Q\,C_2}{g}\cos\alpha_2$$

but,

$$C_2\cos\alpha_2 = U_a + V_a\cos\beta_2$$

(note that $\cos\beta_2$ as in Fig. 1 is minus.)

The work done by the pump in 1 sec. is this force multiplied by the distance, U_a, through which it is overcome in 1 sec. Therefore,

$$\text{Work} = \frac{Q}{g}\,(U_a^2 + U_a\,V_a\cos\beta_2)$$

If H is the total head imparted to the fluid by the pump, then work done $= Q\,H$; therefore, in the writer's Equation (37),

$$H = \frac{U_a^2 + U_a\,V_a \cos \beta_2}{g}$$

In this formula, H is the head delivered by the pump at the periphery of the propellers (line of discharge). It is made up of suction lift, pressure head, and velocity head. It does not include head dissipated between intake to suction pipe and periphery of propellers. If the pump worked with perfect efficiency in all parts (both before and after passing the line of discharge), the work required of the motor would be $Q\,H$ and the lift would be H, as computed by the writer's Equation (37).

O. GRUENBERGER,* ESQ. (by letter).†—The writer's criticism is intended solely for the purpose of helping to clear up any apparent discrepancies in the existing theory of centrifugal pumps.

In explanation of his new theory,‡ the author describes how the sum of three velocity heads, corresponding to three points in the flow through the pump, can be used to calculate the actual head produced with various rates of discharge. It is difficult to understand why these three velocity heads, and only these three, should yield this result. Obviously, another point in the flow could be added and the result would be different. Be that as it may, the author states§ that the sum of the three velocity heads is to be subtracted from the "shut-off" head as found by test. If that is the procedure, no help can be gained from this method.

The author does not state whether the "shut-off" head, found by test, is identical with a calculated shut-off head, although Figs. 9,‖ 10,‖ and 11,¶ indicate that such is the case. This theory also fails to explain a "rising characteristic".

In the first part of the paper, Professor Sherzer states** that the shut-off head can never be more than $\frac{U_a^2}{2g}$. As a matter of actual fact it is usually larger than that. Possibly the explanation for a small increase in head could be found in the fact that the water rotates with the impeller in a radial space of the casing, with a velocity slightly in excess of that of the impeller; but this would hardly account for pressures approaching the value of $\frac{U_a^2}{g}$.

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† Received by the Secretary, November 23, 1927.

‡ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1789.

§ *Loc. cit.*, p. 1792.

‖ *Loc. cit.*, p. 1793.

¶ *Loc. cit.*, p. 1794.

** *Loc. cit.*, p. 1778.

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WATER-POWER APPRAISALS

Discussion*

By Messrs. Walter L. Huber and D. F. McCurrach.

Walter L. Huber,† M. Am. Soc. C. E. (by letter).‡—Water power properties are usually valued for one of four purposes: (1) Rate-making; (2) sale; (3) accounting and capitalization; and (4) taxation. A fundamental question is whether valuation is the same, regardless of purpose. Courts and commissions have often recognized that valuations, or particular elements of valuation, may vary with the purpose.

The so-called method of capitalized net earnings is one which has received consideration since privately owned companies have been engaged in public utility service, but this method has been found to be wholly untenable as a basis for rate-making. This is, of course, evident. Since the value of the property, according to this method, is dependent upon the return which is, in turn, merely a product of the rate, one would, in any attempt to utilize this method for rate-making, be reasoning in a circle. Following this method, an increase in rates would require an increase in capitalization, or a decrease in rates would require a decrease in capitalization.

The writer would hesitate to put much reliance in this method when advising concerning the desirability of purchasing any water power property. It is now an established fact that the public utility commissions in many States have authority to fix rates for sale of power, including the right to abrogate private contracts negotiated to fix such rates. Thus, it is evident that any value derived by a method dependent upon rates is subject to change almost over night and that such a method is not only not applicable to rate-making, but is a rather insecure basis for valuation for other purposes.

Any fair valuation of a water power must be derived from careful consideration of all elements which may affect that valuation. Neither the method of considering the next available source nor the method of considering possible market value can be ignored as the author would imply. The so-called "Steam Substitution Method",§ which is a limited phase of the method of considering the next available source, sometimes has a very real application. When some of the Western water power plants are unable to produce elec-

* This discussion (of the paper by William H. Cushman, M. Am. Soc. C. E., published in October, 1927, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Civ. Engr., San Francisco, Calif.

‡ Received by the Secretary, November 1, 1927.

§ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1843.

trical power at prices as low as it can be produced at the available market (with available cheap fuel oil), the value of those plants is so lowered that little or no value can be affixed by methods of "Capitalized Net Earnings"* or otherwise, regardless of rules of Courts or commissions. A higher law, that of supply and demand, in this case outweighs all arbitrary man-made rules.

A striking example is afforded in Southern California where most of the available water power sites have been developed, but where cheap fuel oil is now abundant. So long as these low prices for fuel oil prevail, such water power sites as now remain undeveloped have but a small measure of value. Their value has also been greatly decreased by recent improvements of steam turbines, which greatly increase their efficiencies, thus lowering the cost of production of electrical energy by steam power. Increased efficiency of steam-generating machinery is here to stay. The price of fuel oil is that of to-day. It is possible that this may be greatly increased in the future and this, in turn, will tend to increase the value of water power sites adapted to serving the same market.

Consideration of market value must exclude consideration of exorbitant and unfair sale prices, but the method is worthy of the careful consideration which it is accorded by the Courts.

D. F. McCurrach,† Assoc. M. Am. Soc. C. E. (by letter).‡—It appears to the writer that, while Mr. Cushman's paper presents a rather fascinating theory on water-power values, at the same time it seems to be somewhat lacking in important details. He assumes in his example that the product of his hypothetical plant is sold at the power plant, whereas it is only in the most unusual cases that this condition would be found, and, where found, the rate of $42 per h. p., used for primary power, appears to be excessive.

In order to sell the energy to consumers, the prospective owners of the project would need to give consideration to the investment in a transmission line and probably step-up and step-down transformers on which annual operation charges, including depreciation, but no return, would very materially deplete the value of the water power shown in Mr. Cushman's paper.

The writer has had occasion to appear before the regulatory bodies of the Pacific Northwest in a number of water-power cases, and he feels assured that the example shown would not be acceptable to the bodies as conclusive evidence of water-power value, where no medium was considered for conveying the power to the prospective consumer.

In that section, the price of $27 per h.p.-year is quite common for large wholesale service and if this rate for primary were applied to Mr. Cushman's example, leaving the secondary figure as quoted, the entire value of the raw water power would be eliminated. The writer's experience has disclosed that minor variations of basic data, which in ordinary computations are not serious, are of major importance in the capitalized net earnings theory and can completely destroy or magnify, out of all reasonable proportion, the values to be derived.

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* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1844.

† Cons. Engr., Portland, Ore.

‡ Received by the Secretary, November 15, 1927.

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THE GREAT YANGTZE BAR

Discussion*

By A. W. Robinson, M. Am. Soc. C. E.

A. W. Robinson,† M. Am. Soc. C. E. (by letter).‡—The writer, having visited Shanghai on several occasions and passed in and out over the bar, is able to visualize the magnitude of the problem and the character of the difficulties so graphically and frankly set forth by Mr. von Heidenstam.

The Board of Engineers' report of 1921§ was largely concerned with the development of the Whangpoo as being Shanghai Harbor proper, and with the re-organization which it deemed necessary to enable the Conservancy Board to attack the great problem of deepening the approaches.

The political condition of the Central Government in China makes it hopeless to expect any aid from that source, as is usual in the United States and other countries, and it is, therefore, on the resources and revenues of the Port itself that the Board must depend for its execution and maintenance. Hence, it is of vital economic importance that the capital cost and maintenance charges of the work should not become so great a tax on the commerce of the Port as to burden or defeat the progress and growth of that commerce which it was designed to promote. This means that if dredging the channel is to be attempted, it must be done on a larger scale and at less unit cost than any other similar work in the world.

The Board of Engineers, in reporting on the subject of approaches, stated:

"The Committee is of opinion that deepening in the South Channel should be undertaken by dredging only, at least in the first instance, and that by this means there is a reasonable prospect of permanent improvement."

It also recommended,

"That the harbor authority should proceed to make and maintain a channel through the Fairy Flats developing it gradually foot by foot to as great

* This discussion (of the paper by H. von Heidenstam, M. Am. Soc. C. E., published in October, 1927, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr., Upper Melbourne, Que., Canada.

‡ Received by the Secretary, November 28, 1927.

§ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1857.

a depth and width as required and possible, and to provide as soon as possible a channel of 600 ft. bottom width for the passage of ships drawing 33 ft. at ordinary neap high water."

This being the decision of this very competent Board of Engineers, the writer will not venture to criticize it or to suggest any alternative. However, there are some very important points to be borne in mind in considering such large scale dredging plants, and in forecasting the probable results that may be obtained. These points are applicable not only to the work in question, but to all similar works throughout the world. Six years have elapsed since that report was made and nothing has yet been done, owing probably to political conditions; but it is presumably becoming more urgent every year.

General Method of Dredging.—In discussing the general method of dredging to be followed, Mr. von Heidenstam is of the opinion* that a broad shallow cut has apparent advantages over a narrow deep one, and that the decision seems to rest exclusively with the drag-suction, self-propelled hopper type of dredge. With this the writer concurs, provided the material be suitable. It is better to make a shallow cut quickly across a bar in the direction of the current, in order that it may be self-scouring as far as possible.

Dredging on the Mississippi River has for many years been successfully done on this principle with a forward feed type of dredge having a wide suction mouth and anchorage lines up stream. The self-propelling drag-suction type of hopper dredge used for deepening ocean bars at New York and other ports also makes a continuous longitudinal cut, although of narrower width. The sand-pumping hopper dredges on the Mersey River, in England, however, work on a different principle, that of dropping the suction down in one place and making a conical depression as the sand runs in. The hopper is filled easily and quickly by this method, and it is supposed that the net result, after the several pockets thus made have evened out, is entirely satisfactory.

It is supposed that so many cubic yards removed from the channel must have the effect of deepening it. While this is undoubtedly true, it is nevertheless a fact that enormous quantities of sand have been removed from the Mersey without corresponding increase of effective depth. This has been attributed to the shifting nature of the sand, but the writer thinks that pocket dredging in this way offers an inducement for the sand to fill up again, whereas in rapid longitudinal cuts there is not the same tendency, and they are, in a measure, self-scouring. In all these localities the material dealt with is sand.

Character of Material.—This has an important bearing on the success or failure of hopper dredging. Sand, especially coarse sand, is ideal for the purpose as it precipitates quickly in the hoppers, filling them to the top while the water runs off in the overflow. However, sand varies much in coarseness and mud content, and more or less solid matter is washed overboard and lost. In very fine sand mixed with mud, as much as 50% or more may be lost in this way; and in alluvial silt or soft mud only some lumps not broken up will remain in the hopper. The loss in this way may become so great that it would not be profitable to dredge it at all by this method.

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1862.

At Brunsbuttel, on the Elbe, the silt is so fine and impossible to precipitate that it comes up in the consistency of soup, and when the hoppers are full the dredge goes to sea without any overflow at all. This means that much water is carried as dead load, but it is better than continuing to pump and losing it overboard.

Precipitation of Solid Matter.—Various methods have been tried to increase the precipitation of solid matter in the hoppers, such as baffle-plates and distributing troughs. In muddy material these are only partly successful, and it is obvious that the two essential conditions necessary to precipitate fine material in suspension are time and quiescence. In a dredge hopper these conditions are difficult to obtain. The incoming stream sets up currents and some agitation and time cannot be allowed for slow precipitation. The best that can be done, therefore, is to distribute the delivery into the hopper so that the least agitation will occur, and design the overflows so that the water is removed after the longest interval between the inflow and outflow both as to time and distance.

These conditions can be much better fulfilled in large hoppers than in small. Hence, a very large hopper dredge offers a better opportunity to reduce the losses in overflow. If the mud will not precipitate in the hoppers sufficiently after pumping, then it will be necessary to consider some other means of filling them.

Type of Drag.—The ordinary form of drag on most of the hopper dredges on the Atlantic Coast (for use in sand) would be quite unsuitable on a packed mud bottom of a nature to require dredging by a bucket or grab. It would simply ride over it and leave a track mark. Such stuff as was picked up and churned by the pump into a fine solution of mud would not precipitate rapidly enough to make a load in any reasonable time.

For such clayey material it would be necessary to excavate it or break it up into lumps suitable to be handled by the pump in such a way that they would not be entirely churned up, but would remain in the hopper. This can be done by a toothed drag or plow having multiple teeth or plowshares, similar to the teeth of a farmer's plow or cultivator. The writer has designed a suction cutter for clay soils, in which multiple cutters or plows are arranged in front of the suction in such a way that all the pump has to do is to pick up the loose material. The cutters are constructed so that if any tooth strikes a snag it will drop back and ride over the obstruction.

Grab Type.—The grab or clamshell type of dredge is well suited for soft clay and the wear and tear and mechanical losses are less than with the pump type. A series of grabs could be arranged on the dredge which could load the hoppers with no loss by overflow and at any desired capacity rate. With electric distribution of power, oil-engine generated, the problem would be simplified and good economy obtained.

Economy of Large Units.—After the dredging the problem is one of transporting and dumping at sea. It is well known that large vessels are more economical than small, both in propelling power and operating costs. The limiting factors in ocean vessels are draft of water and time of loading.

Table 3 was prepared by the writer for another paper.* It shows the comparative economy of cargo vessels of 2 500 to 25 000 tons dead weight capacity.

TABLE 3.—Comparative Economy of Cargo Vessels.

Dead weight capacity, in tons	2 500	5 000	10 000	15 000	20 000	25 000
Length of vessel, in feet	215	300	430	510	590	675
Beam of vessel, in feet	36	44	58	68	72	80
Draft of water, loaded, in feet	22	25	28½	32	34	36
Speed, sea-miles per hour	11	11	11	11	11	11
Indicated horse-power	1 460	1 900	2 800	3 750	5 000	6 000
Coal per hour, in pounds	2 920	3 600	5 000	6 380	7 350	7 950
Coal per 100 ton-miles, in pounds	10.6	6.5	4.5	3.9	3.3	2.9

The dimensions given may be varied, of course, to suit different designs. For instance, if less draft is required the hull may have more length and beam; but the comparison is sufficient to illustrate the great saving in coal consumption in the larger sizes. Thus, a vessel of 2 500 tons consumes 10.6 lb. of coal per 100 ton-miles at 11 knots, while a vessel of 25,000 tons only consumes 2.9 lb. per 100 ton-miles at the same speed. In other words, a given amount of cargo is carried in the larger vessel with 27% of the fuel, and a saving of 73%, with intermediate sizes in proportion.

Size of Plant.—As to size and capacity of such a dredging plant, the writer does not offer any opinion at this time, except to state the possibilities. In the present case the limiting depth of water of 18 ft. could be exceeded at high tide and after the channel has been partly deepened. The vessel, therefore, might be built to carry, say, half load at 18 ft. and full load at 22 or 24 ft., which would permit the use of a vessel of 15 000 tons.

Stability of Channel.—The question of the stability of the completed channel would remain to be proved by time, whether the initial dredging were on a large or a small scale. A tenacious mud would apparently offer greater stability than sand or silt, and it is probable that maintenance costs would not be great. A comparison of annual surveys would indicate the present rate of change.

Report of Board of Engineers.—Perusal of the report of the Board of Engineers mentioned by the author discloses a difference of opinion as to procedure.

One member stated:

"I differ in being of opinion that the data available at present are not sufficient to justify me in making the dredging a definitive scheme of improvement. I recommend the carrying out of an experimental dredging before taking up the actual dredging work, in order to ascertain the feasibility of the latter, and under favorable circumstances, the amount of work needed for the maintenance of a deep water channel on the bar."

The other six members stated,

"The experimental dredging recommended will prove to be futile and it would be unsafe to base any conclusions on the success or otherwise, of the results which might be obtained thereby. They consider it dangerous to

* "Economy in Ocean Transportation," by A. W. Robinson, *Proceedings*, Eng. Inst. of Canada, February, 1919.

follow this proposal for the reason that failure to effect anything approaching permanent deepening would create the impression that a satisfactory channel over the Fairy Flats cannot be produced by dredging. They believe on the contrary, that there is no reasonable doubt about providing a greater depth by dredging, and that the depth so obtained can be maintained by the same means. The one condition they wish to lay down is that the dredging operations should be on a really extensive scale."

The writer thinks that there is some ground for both these contentions. On the one hand some experimentation would seem to be essential to determine the behavior of the material under actual conditions of dredging; the rate of precipitation in a hopper, if pumping is used; the losses in the overflow; the best type of excavating or plowing drag suction to be used; and, generally, whether sufficient material can be retained in the hopper to make the pumping method feasible. If it cannot, a test of some other method should be made.

The material on the bar is described as "mud, fairly easily disturbed by a stream of water, but when consolidated presenting considerable resistance to dredging by grabs or bucket dredges." The author does not state whether any experiments have been made to determine the points mentioned.

On the other hand such an experimental plant would prove nothing as to the general results, in deepening the channel or its permanence, but it would certainly yield useful data. The writer, therefore, is inclined to agree with the Minority Report as to the necessity of obtaining data on which to base a sound design suitable to the conditions before embarking on any large expenditure.

With these data obtained, there is no doubt that a seaworthy dredging plant, that could deal with the large quantities involved at a low cost, could be designed and built with every assurance of success.

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A GRAPHIC METHOD FOR DETERMINING THE STRESSES IN CIRCULAR ARCHES UNDER NORMAL LOADS BY THE CAIN FORMULAS

Discussion*

BY MESSRS. WILLIAM CAIN, WALTER L. HUBER, AND GLEN EDGAR EDGERTON.

WILLIAM CAIN,† M. AM. SOC. C. E. (by letter).‡—Any one who has computed the unit stresses at extrados and intrados, at crown and abutment, for even one circular arch subjected to uniform normal loads, can readily appreciate the enormous amount of labor required to construct the many diagrams given in the author's Figs. 3 to 13.§ Mistakes are easily made for any one example, but they are readily discovered where successive values are plotted. A smooth curve gives confidence in the results. The author's diagrams are very clear and cover every desirable central angle and many ratios, $\frac{t}{r}$ and they should prove of great value to the busy designer. The writer has only checked the values of ϕ_e and ϕ_i, given in Figs. 10 to 13, for $\frac{t}{r} = 0.3$, with central angles ($2\,\phi_1$) of 40°, 90°, and 180°, and has found all of them to be correct.

As it is not evident from the equations given for "thick arches", for a given head and central angle, that the stresses, s_e, s_i, vary only with the ratio, $\frac{t}{r}$, a proof will be given.

By reference to the writer's discussion,‖ it is seen that,

$$\frac{t}{r_n} = \log_e\left(\frac{r_e}{r_i}\right) = \frac{t}{r}\left(1 + \frac{1}{12}\left(\frac{t}{r}\right)^2 + \frac{1}{80}\left(\frac{t}{r}\right)^4 + \cdots\right)$$

* Discussion of the paper by Frederick Hall Fowler, M. Am. Soc. C. E., continued from December, 1927, *Proceedings*.

† Prof. Emeritus, Univ. of North Carolina, Chapel Hill, N. C.

‡ Received by the Secretary, November 3, 1927.

§ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, pp. 1900–1915.

‖ *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 524, Equation (94) and footnote.

Also, since, $I = \frac{1}{12} t^3$ and $K^2 = \frac{1}{12} t^2$, the value of D_n* can be written,

$$D_n = \left(\phi_1 + \frac{1}{2} \sin 2\,\phi_1\right)\left(1 + \frac{1}{12}\left(\frac{t}{r_n}\right)^2\right) - \frac{1 - \cos 2\,\phi_1}{\phi_1} + 0.24\left(\frac{t}{r_n}\right)^2\left(\phi_1 - \frac{1}{2} \sin 2\,\phi_1\right).$$

Thus, since $\frac{t}{r_n}$ can be expressed in terms of $\frac{t}{r}$, D_n can be written as a function of $\frac{t}{r}$ and constants for a given central angle.

It will now be shown that the unit stresses, s_e and s_i, at the extrados and intrados at the abutment section, making the angle, ϕ_1, with the crown section, for a constant head and central angle, vary only with $\frac{t}{r}$. On substituting the value of

$$X = \frac{p_e r_e}{D_n} \frac{\sin \phi_1}{6}\left(\frac{t}{r_n}\right)^2$$

in the values for P_1 and M_1,† the last two formulas of Equation (107) can be written (after reduction),

$$s_e = p_e \left\{ \frac{2 \sin \phi_1}{D_n}\left(\frac{\sin \phi_1}{\phi_1} - \cos \phi_1\right) \frac{\frac{t}{2} + c}{t} - \left[\frac{1}{\frac{t}{r_n}} - \frac{\sin \phi_1}{6\,D_n}\left(\frac{t}{r_n}\right) \cos \phi_1\right]\right\}$$

and,

$$s_i = p_e \frac{r_e}{r_i}\left\{ - \frac{2 \sin \phi_1}{D_n}\left(\frac{\sin \phi_1}{\phi_1} - \cos \phi_1\right) \frac{\frac{t}{2} - c}{t} - \left[\frac{1}{\frac{t}{r_n}} - \frac{\sin \phi_1}{6\,D_n}\left(\frac{t}{r_n}\right) \cos \phi_1\right]\right\}$$

The values of s_e and s_i for the crown can be similarly derived. In fact, these unit stresses may be written down from the last two formulas on replacing cos ϕ_1 by 1 and leaving the other terms unaltered.

On noting that,

$$\frac{\frac{t}{2} + c}{t} = \frac{1}{2} + \frac{r - r_n}{t} = \frac{1}{2} + \frac{1}{\frac{t}{r}} - \frac{1}{\frac{t}{r_n}}$$

$$\frac{\frac{t}{2} - c}{t} = \frac{1}{2} - \frac{r - r_n}{t} = \frac{1}{2} - \frac{1}{\frac{t}{r}} + \frac{1}{\frac{t}{r_n}}$$

and,

$$\frac{r_e}{r_i} = \frac{r + \frac{t}{2}}{r - \frac{t}{2}} = \frac{1 + \frac{t}{2\,n}}{1 - \frac{t}{2\,r}}$$

* *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 531, Equation (104) and footnote.

† *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1897.

and since $\left(\frac{t}{r_n}\right)$ has been expressed in terms of $\left(\frac{t}{r}\right)$, it is seen, that for a given p_e and ϕ_1, s_e and s_i vary only with $\frac{t}{r}$. This is not true for P_0, M_0, P_1, and M_1, but it is true for s_e, and s_i at either crown or abutment. It is thus seen that the same values are derived for $t = 15$ and $r = 50$, as for $t = 30$ and $r = 100$, since $\frac{t}{r} = 0.3$ in both cases.

In using Equation (107),* any convenient values of t and r that have the assumed ratio, can be used. The values of s_e and s_i thus derived can be checked by the formulas for s_e and s_i just given. Evidently, these formulas furnish the shortest solution for the unit stresses.

In using them, for assumed values of t and r, giving the required ratio, it is best to compute c from Equation (95),†

$$c = r - r_n = r - \frac{t}{\log_e \left(\frac{r_e}{r_i}\right)}$$

and thus derive, directly,

$$\frac{\frac{t}{2} + c}{t} \text{ and } \frac{\frac{t}{2} - c}{t}$$

Also, $\frac{t}{r_n} = \log_e \frac{r_e}{r_i}$ is directly computed without the use of the infinite series.

As Mr. Fowler, in drawing his diagrams, has assumed a 10-ft. head,

$$p_e = \frac{10 \times 62.5}{144} = 4.34 \text{ lb. per sq. in.}$$

to agree with the diagrams. The stresses vary directly as p_e, or as the head.

The formulas for s_e, s_i, reduce to those for a prismatic beam on assuming, for t finite, that r increases indefinitely, so that $c = 0$, $r_n = r$, and $p_n = p$. This will be evident if, in the usual formula for the unit stresses in a straight beam,

$$\frac{P}{t} \pm \frac{6\,M}{t^2},$$

the values of P and M‡ are substituted; when, on reduction, the previous values for s_e, s_i, corresponding to $c = 0$, $r_n = r$, and $p_e = p$, are derived.

In deriving the formulas for thick arches, the writer ignored the work of the radial stress, as its influence was found to be small.§ Also, Poisson's ratio was assumed equal to zero. This ratio is easily included, as B. F. Jakobsen, M. Am. Soc. C. E., has shown.‖ His pioneer work on this subject, and the effect of water-soaking, are both important contributions. However,

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1897.
† *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 525.
‡ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1895.
§ *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 538.
‖ *Loc. cit.*, p. 488; also, see "Example," p. 539.

it may be remarked, that the experiments for determining Poisson's ratio have been made on very small specimens and it is very doubtful if the ratio thus found applies to arches 30 to 50 ft. thick.

In addition to this uncertainty, there are others that militate against the hope of an exact solution, such as the unknown shrinkage of concrete in drying out, the changes in E with stress and time, the effect of plasticity, water-soaking, and the elastic yielding of the foundation and abutments. Besides, there is doubt about the uniformity of the water pressure on a horizontal arch, and as to the relative proportion of the water pressure carried by the arch and the cantilever; so that the computation of stresses for the given hypotheses, which are so greatly facilitated by the author's diagrams, although a necessary step, is not the complete solution, for the engineer must consider all the influences mentioned in the design of arch dams. Mr. Jakobsen states* that "he had hoped to install some resistor type of cartridges for measuring deformations in both the Pacoima and the Big Santa Anita Dams, but was unable to obtain the necessary funds". This is greatly to be regretted. Experiments on thin arch dams that buckle and crack are valuable for such conditions, but they do not represent the conditions for thick dams where the device of grouting to avoid cracks has been used and where buckling is not experienced. Experiments on such dams will tend to clear a number of doubtful points, and it is to be hoped that they may be instituted in the future.

WALTER L. HUBER,† M. AM. SOC. C. E. (by letter).‡—The writer has made much practical use of the diagrams§ prepared by Mr. Fowler. Before accepting them as a substitute for the more laborious methods of exact calculation he made some comparisons of the two methods. The difference is, in every case, very small, being much less than various uncertainties of assumptions governing practical designs. William Cain, M. Am. Soc. C. E., through his formulas, made a very valuable contribution to engineering literature. Mr. Fowler has made this contribution readily available to the designer of arch dams and for this service deserves the thanks of all members of the profession engaged in designing structures of this character.

GLEN EDGAR EDGERTON,‖ M. AM. SOC. C. E. (by letter).¶—The curves presented by Mr. Fowler should prove to be of great assistance to engineers engaged in the design or investigation of dams of the arch types. The increasing number of structures of this type that have been erected in recent years and the indisputable economies made possible by them when circumstances are favorable, indicate that more and more frequently in the future, engineers will be required to consider arch dam design, even, perhaps, in many cases in which other types may finally be adopted.

Designs for arch dams that show reasonably conservative unit stresses under analysis by the Cain formulas have been approved in several instances

* *Transactions*, Am. Soc. C. E., Vol. 90 (June, 1927), p. 586.
† Cons. Civ. Engr., San Francisco, Calif.
‡ Received by the Secretary, November 1, 1927.
§ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, pp. 1900–1915.
‖ Maj., Corps of Engrs., U. S. A.; Chf. Engr., Federal Power Comm., Washington, D. C.
¶ Received by the Secretary, November 18, 1927.

by the Federal Power Commission, and by State and local public officials as well. To that extent, at least, these formulas may be considered to have become established as acceptable standards in practice as well as in theory.

Without attempting to make a general check of the curves presented by Mr. Fowler, several designs previously analyzed by the Cain formulas and computation methods have been investigated by means of them. The critical stresses calculated by the two methods agree very closely. Figs. 3,* 4,† 5,‡ and 6§ were used. The central angles for these calculations ranged from 84° 44′ to 134° 0′; $\frac{t}{r}$ from 0.0448 to 0.312 (the latter value requiring extrapolation); and the head from 25 to 185 ft. The maximum difference between the unit stresses determined by the two methods amounted to 19 lb. per sq. in.

Mr. Fowler has made a valuable contribution to the engineering art in developing these time-saving charts..

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* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1900.
† *Loc. cit.*, p. 1901.
‡ *Loc. cit.*, p. 1902.
§ *Loc. cit.*, p. 1903.

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PAPERS AND DISCUSSIONS

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SURVEYING AND MAPPING IN THE UNITED STATES

Discussion*

BY MESSRS. T. P. PENDLETON AND C. L. HALL.

T. P. PENDLETON,† ASSOC. M. AM. SOC. C. E. (by letter).‡—This interesting paper presents very concisely the history of surveying and mapping in the United States, in so far as it relates to National surveys. It shows that the first crude surveys of large areas were made in the West and were largely of an exploratory nature. It is interesting to learn that the first contour map in the United States was made in 1835, although it was not until after the Civil War that the method was extensively adopted. It was only natural that many methods were tried and discarded in the search for that which presented the most advantages. A description of the various methods that were used in the days of the Hayden, Wheeler, Powell and King surveys would make interesting reading, as would no doubt the personal experiences of these pioneers in the art.

Maps that are classified as small scale would have been called large scale by these early surveyors, and the accuracy that is now obtained would probably seem to them sufficient for many years to come. However, the need of the country continually calls for better and better maps, and that this need is recognized by the Geological Survey is clearly shown by a casual inspection of maps made at ten-year intervals since the first appropriation for topographic surveys was made in 1888.

The United States not only needs better maps, but more of them. This need can be met by existing organizations and bureaus on short notice whenever Congress realizes that ample appropriations for this purpose are an investment that will make big returns to the country. It is necessary to look into

* This discussion (of the paper by C. H. Birdseye, M. Am. Soc. C. E., presented at the meeting of the Surveying and Mapping Division, Philadelphia, Pa., October 6, 1927, and published in the October, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Topographic Engr., Brock & Weymouth, Inc., Philadelphia, Pa.

‡ Received by the Secretary, November 23, 1927.

the future in mapping, for it is for future needs that maps are made. The safest policy is to supply maps as good as can be made at the moment, for there is little doubt that unless this is done they will be found inadequate in the not distant future. A well-made map should be constructed on a framework consisting of accurate triangulation or traverse to assure reliable map positions. The contours and elevations should be dependent on near-by precise level lines; all other level lines should be so carefully run that it will not be necessary to make adjustments for any but very small circuit closures; and, finally, the contour lines should be located so well that any check that can be applied to them will indicate that they are not incorrectly located more than a short distance. This last requirement is admittedly difficult when surveys of brushy or timbered areas are made by methods now commonly in use, but recent tests of maps made by aerial photography prove that such terrain is no longer impossible to map with accuracy.

A strict observance of the conditions mentioned will assure a map that can be revised from time to time at small cost and kept as up to date as are mariners' charts. The map then becomes a living thing, as it were, that is constantly changing as the country grows. Unfortunately, such revision cannot be made unless the map, as first constructed, is sufficiently accurate to serve as a base to which new cultural features can be added. To revise a map successfully, it should only be necessary to add the new features and delete those that have disappeared, but not to change other details. It is impossible to use an inaccurate map as a base for revision if satisfactory results are expected, as the changes that occur due to expansion of industry and growth of population have a perverse habit of manifesting themselves in weak spots of old maps. That re-survey and not revision is needed in the case of many Government maps is appreciated when their method of construction and the cost of the original surveys are investigated. Fine maps, like most other things, cannot be obtained for "a song", and unless they are well made in the first place they will be a continual source of expense for maintenance, and of loss to industry due to their inherent errors.

The author shows* that this country might have had adequate topographic maps of the public lands, had it not been for a lack of foresight in the days when this work was inaugurated. The loss due to the inaccurate early public land surveys was immeasurably increased by the fact that this splendid opportunity to secure topographic maps of all the public domain was entirely overlooked. The country is still paying the cost of that mistake.

About 57% of the Continental United States, exclusive of Alaska, has never been mapped even in the roughest manner. What the percentage may be that is mapped accurately enough to justify revision, it is difficult to state, although it can be said without hesitation that it is small. For these reasons it is to be hoped that the Temple Act providing for the completion of the topographic map of the United States will soon be put into effective operation, and that the maps produced under this Act will be of such a high standard of accuracy that they will serve satisfactorily for many years. The

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1920.

entire cost of all maps made by the U. S. Geological Survey has undoubtedly been returned to the country many times, but this usefulness and the corresponding savings would surely have been greatly increased had their accuracy been greater. Who can say how many times their cost would have been repaid had accurate maps of the Mississippi Valley been in existence during 1927? What would they now be worth for the study of river control?

Observations by the writer indicate that the general purpose map, on the scale 1 to 62 500, is widely used for general studies and for studies of many special problems of relatively small areas by engineers in the business world. In order to use these maps in such studies, it is necessary for the engineer to enlarge the map so that the proposed structure will be of appreciable size and can be properly visualized in relation to other features. This introduces a grave difficulty, as the enlargement is always much greater than the field scale used by the topographer in the original survey. Consequently, his map suffers by comparison with even a crude map made on a larger scale. It seems that by increasing the original map scale it would be possible greatly to extend the usefulness of these small scale maps without changing the publication scale or greatly increasing the cost. This increase in accuracy is worthy of consideration, for not only would the value of the maps be enhanced for present needs, but it would assure their satisfactory revision when this is needed.

The author fully appreciates the possibility that future mapping will be done by aerial photographic methods to a large degree. As he states,* such methods will probably never entirely replace ground surveys; nor is this desirable, as place names, political and property boundary lines, and a certain amount of fundamental control will always have to be obtained on the ground. In certain areas it may even be necessary to gather other essential data, although it should be possible to so arrange the field work that this may be obtained when other necessary information is secured.

The accuracy and ease with which contours may now be drawn by stereoscopic methods in timbered country is a never-ending cause of surprise, and to attempt to state what will eventually be done by this method in such areas is impossible.

In his comments on the use of aerial photography,* the author undoubtedly had in mind the method now used by the U. S. Geological Survey, in which planimetric detail only is taken from the photographs and the contours are added in the field. Otherwise, it is difficult to account for his statement that streams must be traversed to determine their fall when photographic methods are used. It can confidently be said that any method of applying aerial photographs in mapping, that fails to make use of image displacements caused by ground relief, omits the most valuable aid that such photographs can render the topographer. This partly accounts for the unsatisfactory results that were obtained in 1925 near Grand Valley, Colorado, for there is no reason to believe this area would have offered any difficulty had it been photographed for mapping by stereoscopic methods.

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1926.

The writer quite agrees that there is little chance for small scale maps to be produced at lower cost in the future, as the demand for increased accuracy will offset any new economy in operation. For this reason, aerial mapping methods that can promise greater accuracy without a great increase in cost should be judged by comparing their cost per square mile with those of ground surveys of equal accuracy, and not by a direct comparison of costs per unit of area.

C. L. HALL,* M. AM. SOC. C. E. (by letter).†—As is to be expected of an eminent topographical engineer, Colonel Birdseye has written a complete and very interesting discussion of topographical mapping in the United States, so far as it could be brought within the limits of the space presumably allowed him. There seems to be but one assertion on which critical comment is in order.

Colonel Birdseye divides the history of mapping into two major periods:‡ One in which mapping was performed "for military purposes", and the other in which it was performed "for economic or engineering uses". These periods are not, however, as the author seems to indicate, epochs in world history, nor even in National history, since they are of radically different duration in different parts of the world. A considerable portion of the mapping now going on in the world, including some being paid for by the United States, is intended primarily for military purposes. In France and Great Britain, while the economic importance of maps may now exceed their military importance, surveys are still conducted under military control and largely by military personnel. On the other hand, in pre-war Germany, as one example, intensive surveys were often conducted by civil personnel under civil control, primarily for military purposes. It is believed that the military value of a map of an area exceeds its economic value if there is a good chance of a war occurring in that area before the map becomes hopelessly out of date. The major periods of the mapping history of the United States named by the author are, in reality, based on the good fortune of this country in keeping its territory—outside of a few Indian areas—clear of the horrors of war since 1865.

Colonel Birdseye's excellent review§ of the present status of the art of aerial surveying should be of special interest. Particular attention should be drawn to his statement that in the revision of old surveys, made necessary by changes in the works of Man, aerial photographs would make it possible to publish an up-to-date map without the necessity of re-surveying by ground methods. This use of the aeroplane in surveying points the way to the possibility of keeping topographical maps up to date with reasonable economy. If any constant demand could be stimulated among the general public for up-to-date topographical maps, the political authorities of the nation would doubtless provide the comparatively small sums necessary for the annual revision of maps of the more quickly changing areas. The road map maker has to revise his maps annually; the commercial atlas publisher has to revise his atlas

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† Received by the Secretary, November 30, 1927.

‡ *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1919.

§ *Loc. cit.*, pp. 1925–1927.

annually; and the Post Office Department has to revise its maps at frequent intervals. The U. S. Geological Survey should not be the only mapping agency in the world required to print re-editions with culture so out of date that often the map is of interest only to local historians.

Colonel Birdseye properly calls the attention of the profession to the backwardness of the United States in mapping activities. This situation, while probably known to most engineers, does not seem to have appealed to them as a matter requiring urgent representations to the political authorities, National and State. This National ignorance of topography has recently become a matter of some interest to the general public.* The basic law which should ensure the mapping of the country has been passed.† All that is needed is popular demand produced by educated leadership which is also enthusiastic. This leadership should be supplied by the Engineering Profession.

* Editorial, *Saturday Evening Post*, October 15, 1927.

† Act of February 27, 1925, Chapter 360, 68th Cong., 2d Sess.

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SURVEYS ON THE COAST OF NEW JERSEY

Discussion*

By C. V. Hodgson, M. Am. Soc. C. E.

C. V. Hodgson,† M. Am. Soc. C. E. (by letter).‡—Three factors necessary to a satisfactory survey have been admirably stressed by Mr. Gelineau, these being adequate accuracy in angle and length measurements, permanent monumentation, and connections to the National system of triangulation surveys. All three factors are important to any survey, but the last two are especially so in regions where the boundary monuments are subject to disturbance by natural causes or by the hand of Man. A slight amplification of these three points may be warranted.

Accuracy.—Mr. Gelineau states that the three purposes which the survey of beach lands must serve, demand that the principal lines of traverse or triangulation be in error not more than 1 part in 10 000, or 1 ft. to 2 miles. This corresponds to what is known as second-order accuracy in Federal surveys. At first thought this accuracy may seem excessive until it is recalled that one object of the New Jersey survey is to define the boundaries of riparian grants, and that the beach lands have a very high value.

For years the U. S. Coast and Geodetic Survey was criticized for what was considered to be the excessive accuracy of its coastal surveys—a specified error of not to exceed 1 part in 5 000 for its main lines of third-order triangulation. It now appears that for many purposes that accuracy is not sufficient, although ample for charting and mapping. It is doubtful, however, if the surveying agencies of the Federal Government would be justified in increasing the accuracy of its detailed control surveys at the present time. Greater accuracy may frequently be needed for cadastral or improvement surveys, but they usually lie outside the province of the Federal Government.

* This discussion (of the paper by Victor Gelineau, M. Am. Soc. C. E., presented at the meeting of the Surveying and Mapping Division, Philadelphia, Pa., October 6, 1926, and published in October, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Asst. Chf., Div. of Geodesy, U. S. Coast and Geodetic Survey, Washington, D. C.

‡ Received by the Secretary, November 5, 1927.

An accuracy represented by a maximum error of 1 part in 10 000 in determined distance is neither difficult nor expensive to secure if proper instruments and methods are used. Where an extensive area is involved, or where there is a strong probability that the value of the land will increase considerably in the future, the accuracy specified by Mr. Gelineau is by no means excessive.

Monumentation.—To monument a survey permanently in a region subject to erosion is a practical impossibility. It is also very difficult to establish monuments which can be easily recovered in a region where accretion or sedimentation is noticeable or where movements of sand dunes are changing the surface of the ground. Various expedients have been tried by the Coast and Geodetic Survey in regions subject to these shore-line changes, including deep sub-surface marks and cypress or cedar posts as witness marks. The present practice is not to mark the station permanently if it is in a location subject to erosion, but to reference it by two monuments placed far enough back from the shore to be secure for a number of years and yet near enough to serve in relocating the old station at any time, with an error of not more than about 1 in.

If the coastal control is by the triangulation method and a line of stations has been established inland on solid ground in locations easily visible from the coast, the preservation of the outer line of exposed station marks is not so important, for they can be relocated at any time from the inner line of stations. In localities where sand drifts or sediments may cover up the marks, witness posts are used in connection with fairly large concrete monuments at the station. These monuments can be readily located by prodding with an iron bar through a considerable depth of mud or sand.

Where the ground is stable it is economy to establish permanent marks. A slight increase in the cost of monumentation may double the period of years during which the survey may be retraced. Underground marks separate from the surface marks, and substantial reference marks placed in property lines, will perpetuate a survey for a long period of years at slight additional expense.

Connection to National Datum.—There are two ways in which a survey may be connected to the North American Datum. By the first method one or more control stations on the National datum are tied in by traverse or triangulation to the new survey, but are used simply as reference points. By the second method the origin of a local system of plane co-ordinates is either made to coincide with a station on the National datum or is connected to it. The latter system is usually followed when detailed surveys are to cover a considerable area. Either method has the effect of making every control station in that region which is on the National datum a reference mark for the new survey, and is the surest method of perpetuating it. If the connection to the National datum stations is made part of the legal record, there is little doubt that any Court would give such evidence great weight in any retracement surveys.

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THE DELAWARE RIVER FROM PHILADELPHIA TO THE SEA

Discussion*

MESSRS. HARRY TAYLOR AND F. C. WARNER.

HARRY TAYLOR,† M. AM. SOC. C. E.—The question of the relative cost of Government and contract dredging has been raised. The difference in cost of the two methods is explained at least partly by the fact that the Government owns comparatively few dredges and these are busy all the year. If work could be let in large contracts, which would keep all dredging contractors' plant busy for long periods and the contractors could be sure that when one job was finished another job would be available, so that all their plant would be always busy, the work should cost less; but whether the price to the Government would be less is another question, for experience has shown that low prices are obtained when there is little work and strong competition. However, such a happy condition for the contractor can never exist, for no contractor would be satisfied with the amount of work he had, and every one would be building more plant in the hope of getting more work and making a little more money.

Another item that favors the Government plant is that the Government does not have to go out and look for work, and is, therefore, not under the expense of bidding for work. Every contractor must bid for every job he obtains, and every bid costs money whether or not a job is obtained. Not every contractor obtains work every time he bids, but he must charge up as part of his overhead expense the cost of preparing his bids, and some job must pay for it or otherwise the contractor will soon go out of business. Still another item is the expense of moving plant from one point to another. These items explain, in part at least, why the Government should be able to do work at a lower price than the contractor. Other items which add to the contractor's

* Discussion of the paper by F. C. Boggs, M. Am. Soc. C. E., continued from December, 1927, *Proceedings*.

† Maj.-Gen., U. S. A. (*Retired*), Washington, D. C.

cost are depreciation and insurance. Depreciation is fully covered by the present cost-keeping methods of the U. S. Engineer Department, but owing to the very excellent care taken of all Government dredges, they last for many years and in some cases are still at work after their entire cost has been charged off. Therefore, the item of depreciation, when such plant is used, is a book charge only, and does not represent actual outlay of Government money. An example may be shown by the dredge, *Manhattan* (Fig. 6*), which was built in 1908 for use in digging the Ambrose Channel. The whole cost of that dredge was charged against the Ambrose Channel improvement, but it was sold to the Philadelphia District and now, after about twenty years, it is in fair condition and still doing good work. Insurance is charged in the cost-keeping accounts for the purpose of making comparative estimates of cost, but that item is a very small one.

The U. S. Engineer Department has adopted the policy of retiring its old, out-of-date dredges. The first sea-going hopper dredges were built with wooden hulls and some of them were used for as long as thirty-five years, but they have finally all been retired from service and most of them have been sold for very small prices, practically junk. All the dredges in use at present have steel hulls, and there is no reason why they should not last longer than those with wooden hulls.

The sea-going hopper dredge does work in certain places, such, for example, as the ocean bars of many harbors, at a much less cost than such work can be done by any other type of dredge. The type of sea-going hopper dredge in use in the United States has been developed entirely by the U. S. Engineer Department. Dredges somewhat similar to the American sea-going hopper dredges are in use in Europe, but the type apparently most favored there is one which was tried in the early days of the Ambrose Channel improvement and proved not to be a success. The newer sea-going hopper dredges in use by the Engineer Department are very expensive, costing $600 000 and more. The great cost of these dredges is probably the reason why contractors do not use them, because they cannot afford to tie up so much money in a single piece of plant that may not be kept busy at all times. The extremely small cost at which these dredges can do work fully justifies the ownership and operation of a limited number of them by the Government. Their great flexibility in being able to work in crowded channels, such as those in New York Harbor, and the facility with which they may be transferred from one locality to another, also give added reasons for their use by the Government. The element of portability is shown by the fact that one of these dredges, which was built on the Delaware River, was sent to Alaska under its own power. There it worked under conditions that would have been very difficult for any other type of dredge on account of the heavy tide, the distance from base of operations, and other unfavorable conditions. The District Engineer, basing his estimates on the best data that he had been able to obtain from contractors as to prices they would probably charge for this work, considered that the use of the hopper dredge saved more than $100 000 to the Govern-

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1979.

ment in the cost of this one operation alone. Considering the difficulties of doing the work with any other type of dredge, it is believed that this estimate of saving is not too high.

F. C. WARNER,* M. AM. SOC. C. E.—The Mitchell and Giesler studies, mentioned by the author,† were made when the Delaware River was in its natural condition, no work of improvement having been done beyond a small amount of dredging. There is no record so far as known to the speaker that the results of these studies had been used in any way in connection with the improvement of the Delaware River until they were taken up by him in his study in 1917.‡ The natural regimen of the river has been materially changed by dredging and the construction of dikes and jetties in connection with the projects for the 30 and 35-ft. channels, particularly in changes in currents, depths, and tidal conditions. It has been found that the mean tidal range has decreased about 0.7 ft. and the elevation of mean low water has been raised about 0.8 ft., compared with the range and elevation of the tides in the river in its natural condition. This is due largely, it is believed, to the improvement work. The existing condition of the river in 1916 rather than its natural condition was, therefore, taken as the basis for this study.

In the author's reference‡ to the study by H. F. Flynn, M. Am. Soc. C. E., in 1918, it is noted that Mr. Flynn takes exception to this, as he believed that the natural regimen of the river should have been used instead of an artificial state. For the reasons stated, the speaker does not agree. For this study, cross-sections of the river channel were taken, in the fall of 1916, at each nautical mile, beginning at Fort Mifflin Light and extending a distance of 40 nautical miles down the river, the locations being the same as for those used in the Mitchell study. Fig. 9 shows the results of this study.

The survey could not be extended the full 46 nautical miles covered by the Mitchell study, nor could intermediate cross-sections be taken, as the field work was stopped by cold weather and ice in the river. To cover the full 46 nautical miles, cross-sections were taken from a recent survey. Examination of many charts and surveys of the river showed that at certain stretches, from 1 mile to 3 miles long, natural depths of 35 ft. or more at mean low water and a bottom width of about 800 ft. existed in both its natural and improved condition. Other stretches were found to be 35 ft. or more deep, but less than 800 ft. wide. These natural depths of 35 ft. or more that existed in the original condition of the river, and still exist at the same places in its improved condition, must be mantained according to some definite law, and it was assumed, therefore, that cross-sections could be accepted as "standard" for the volume of tidal flow at these points, from which theoretical cross-sections for natural maintenance at intermediate points could be determined. With this in view three typical, or "standard", cross-sections were selected, 3, 20, and 44.6 nautical miles from Fort Mifflin Light, respectively, the last being at the head of the Delaware Bay channel where there was a depth of 35 ft. at

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† *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, pp. 1968–1969.

‡ *Loc. cit.*, p. 1970.

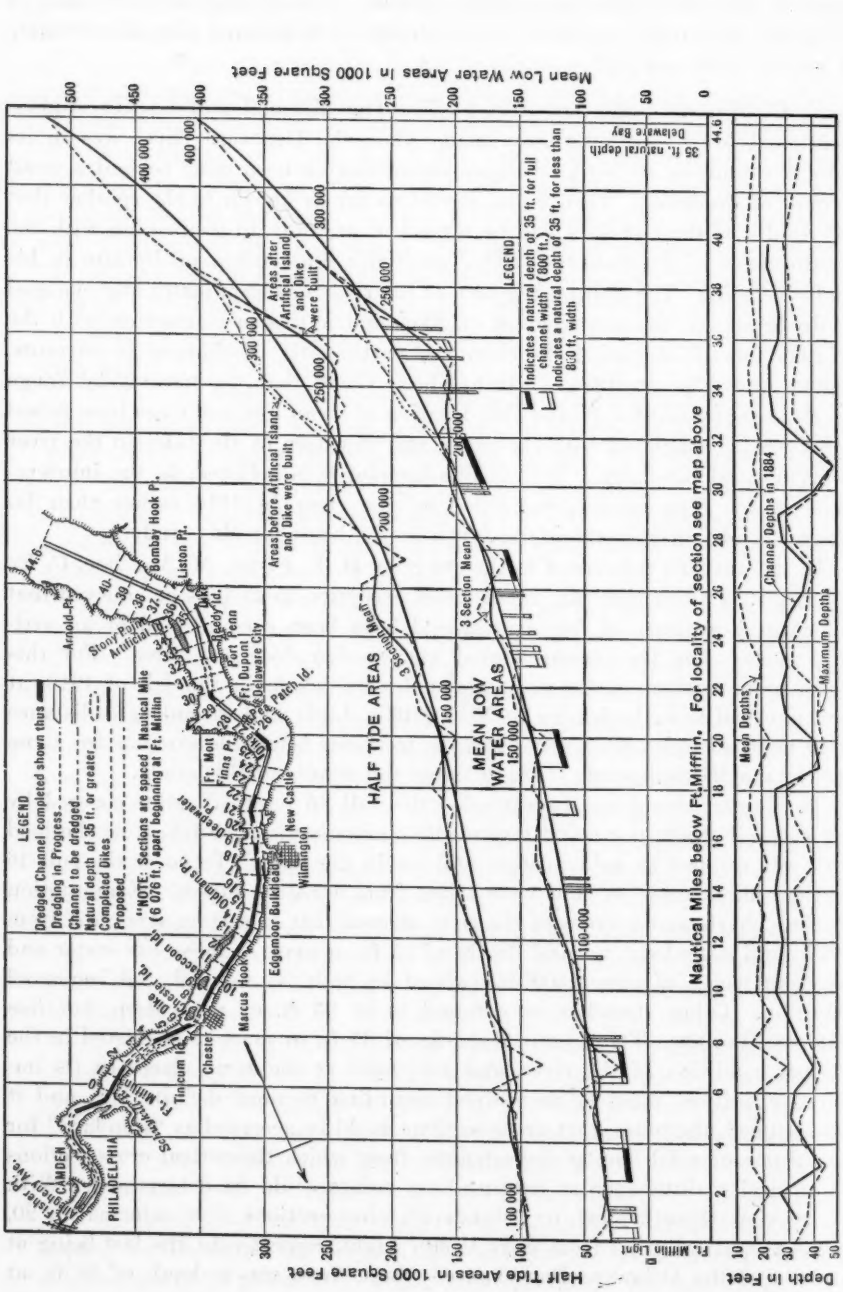

FIG. 9.—CURVES OF CROSS-SECTIONAL AREAS AND DEPTHS, 35-FOOT CHANNEL, DELAWARE RIVER.

mean low water and a bottom width of 800 ft. For a better average cross-section at these points the arithmetical mean of three cross-sections, one above and one below, was used. Applying the Giesler equation, the hyperbolic curves for cross-sectional areas at mean low-water and half-tide stages were determined, from which intermediate cross-sectional areas at each nautical mile were computed.

The author's Table 4* gives the half-tide cross-sectional areas for each 5 miles from 5 to 40 nautical miles, inclusive, below Fort Mifflin Light. These curves, thus determined, platted on the low-water and half-tide profiles of cross-sectional areas obtained from the survey, are shown on Fig. 9. The low-water curve passes through five cross-sectional areas having natural depths of 35 ft. or more at mean low water, all of them being 800 ft. wide, including two "standard" cross-sectional areas used in determining the curve. They have a length of 7 nautical miles out of the total of 44.6 nautical miles under consideration. The curve also passes through, or close to, other intermediate points having a natural depth of 35 ft. at mean low water, which serves to verify the correctness of the equation. It appears safe to assume, therefore, that as this curve passed through five "standard" cross-sectional areas with an aggregate length of 7 nautical miles and the "standard" cross-sectional area at the head of the Delaware Bay channel at Nautical Mile 44.6, it should give correct theoretical cross-sectional areas for the natural maintenance of a 35-ft. channel, 800 ft. wide at all points of the remaining 37.6 nautical miles of channel of less than 35 ft. depth.

From these theoretical cross-sectional areas, widths at mean low water may be determined at each point where a reduction in the present cross-section and width is required. It is believed that this theory is entirely rational and can be applied, in connection with the location of regulation works for natural maintenance, at sections of the river channel that now require continuous maintenance dredging. The author has quoted† the speaker, as to the practical application of this theoretical law.

The speaker intended to continue this study progressively by taking cross-sections annually at the same points and also study the tides, currents, river slopes, etc. With this in view automatic tide gauges were installed in the spring of 1917 at five places below Fort Mifflin Light to the head of Delaware Bay. Precise level bench-marks of the U. S. Coast and Geodetic Survey and the U. S. Engineer Office at Philadelphia, Pa., are located along the west shore of the river, to which the planes of mean low water, as determined by these gauges, may be referred. This program was interrupted, however, by his transfer to another district.

The Delaware River carries a large percentage of material in suspension, brought down from its headwaters and from its many tributaries, particularly during periods of freshets, much of which is deposited in the dredged channel. It is believed that this sediment is precipitated largely when the fresh water, with its suspended burden of silt, meets the salt water and this great mass of mud and water moves along the bottom up and down with the

* *Proceedings*, Am. Soc. C. E., October, 1927, Papers and Discussions, p. 1971.
† *Loc. cit.*, p. 1970.

tides until a condition favorable to its deposit is reached, which is usually at the broad sections of the river where the currents slow down. These are the sections that require continuous dredging for maintenance. It is believed that the degree of salinity of the water has much to do with the precipitation and deposit of sediment, but as no study of this feature has been made, so far as known, the particular sections where this occurs are not known. It is worthy of investigation. It is known, however, that the degree of salinity at any particular section of the river changes materially in dry and freshet seasons.

The quantity of the annual deposit of sediment in the limits of the 35-ft. channel from Philadelphia to the head of Delaware Bay was estimated in 1915 by the U. S. Engineer Office, at Philadelphia, at 17 000 000 cu. yd. This is a maximum estimate but it cannot be far from correct as maintenance dredging during the two years ending June 30, 1926, and June 30, 1927, amounted to 9 200 586 and 10 845 001 cu. yd., respectively, and a controlling depth of only 34 ft. was obtained. This presents a serious problem for the maintenance of a 35-ft. channel at a reasonable cost. Certain sections require continuous dredging by sea-going hopper dredges from which the dredged material is dumped in basins and rehandled with pipe-line dredges and deposited on shore, or in bulk-headed basins. This process must be continued until a system of regulation by contraction of the width for natural maintenance is adopted. The construction of longitudinal dikes at these sections providing the proper width and cross-section of channel, as determined from theoretical and practical studies, will not only assist natural maintenance, but will provide areas for the deposit of dredged material within the reach of pipe-line dredges and will obviate the necessity for rehandling.

The provision of these areas for depositing dredged material in one operation, together with the reduced amount of dredging that will be required for maintenance as a result of a system of regulation for natural maintenance, will more than pay the cost of construction of the dikes and purchase of land and riparian rights in connection therewith.

The excavation and maintenance of a 35-ft. channel, and of a 40-ft. channel (which is now under consideration, and which will be required in the not distant future in the Delaware River) at a reasonable cost is a greater engineering problem in river and harbor work than in any other tidal river on the Atlantic or Pacific Coasts. It is believed that this matter has not been as thoroughly understood and discussed by engineers as its importance warrants, and it is hoped that this paper will be the subject for discussion of the problems involved, that it will result in definite methods for regulations and a much lower cost for maintenance.

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SNOW-REMOVAL METHODS IN NEW YORK, N. Y.

Discussion*

By Messrs. William Goldsmith and R. A. MacGregor.

William Goldsmith,† M. Am. Soc. C. E.—Mention has been made of snow removal work in large cities. In the business sections of smaller cities the snow removal problems are probably the same. Yonkers, N. Y., which may be called a small city (population of 120 000), has a business section and also a residential section. It is, however, as small cities go, what may be called a suburban community and, as such, its citizens have to be catered to in a different way than is the case in a large city like New York. When perhaps 50% of the population are commuters who work in New York and sleep in Yonkers, their demands are more urgent on account of the need of immediate access to the railroad stations. Each one wants the road from his house to the railroad station plowed for him immediately after the snow falls.

In order to fill this want, which is really an important one, a system was inaugurated in 1926 that produced remarkable results. The city was subdivided into ten districts. Each district was mapped and the route of the snow plow shown on it, indicating exactly the order in which streets were to be plowed and the direction in which the plow was to go, keeping in mind that access to the railroads, schools, and the business sections were of first importance. In order to make this plan practical the man who did the mapping went out on the job with the driver of the snow plow and in this manner was able to determine whether any practical changes should be made.

It was found that the best results were produced by a caterpillar tractor with a V-shaped plow in front of it; and although the city covers about 22 sq. miles and has 130 miles of streets, a path was plowed over each street within 24 hours after a snowfall, the plow being sent out as soon as 3 in. of snow had fallen.

This is a residential area pure and simple, where the sidewalks are not cleared of snow by the City. Immediately after a storm the people use the

* This discussion (of the paper by Elmer C. Goodwin, Assoc. M. Am. Soc. C. E., presented at the meeting of the Highway Division, New York, N. Y., on January 20, 1927, and published in October, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Commr. of Public Works, Yonkers, N. Y.

center of the road where the plow has opened a way. Of course, in the business sections, the snow is dumped into sewers and carted away with snow loaders, etc., the same as in New York. Referring to New York, it was the speaker's privilege in 1912 to make experiments on its sewers to ascertain how long it takes for snow to melt when it is dumped into a manhole. It was the result of these experiments that made possible the enlargement of the method of using the sewers for the disposal of snow.

The experiments showed that no matter how much snow was dumped into a sewer manhole, it would be melted within 200 ft., or before it reached the next manhole. This agrees with theory when it is understood that the average temperature of sewer water in Manhattan is 60° Fahr. and there is enough of it present in sewers more than 10 in. in diameter to convert all the snow into water. Incidentally, ice was melted in this manner about as quickly as snow. During the period of these experiments many inventors came forward with different ideas for melting snow and doing other things that would be cheap and effective ways of ridding the city streets of snow. Thus far, none of these has proved successful and, in Yonkers, no attempt has been made to use the untried methods for snow-removal purposes, but the sewers are used wherever possible.

In this suburban community the plan of immediately plowing the roadways for pedestrians and vehicular traffic has demonstrated that it is really worth while. The expense, outside the purchase of equipment, is very small, and the taxpayers are very much elated at the service given.

In 1926, there was a little more snowfall in Yonkers than in the City of New York. Business sections were cleaned the same way in both places, the snow being dumped in sewer manholes and at the river-front. By many, it was said that Yonkers was cleaned up much more quickly than New York City, and it cost approximately $79 000 to do it. In comparison with New York City's expenditure, this seems to be a "drop in the bucket".

R. A. MacGregor,* M. Am. Soc. C. E.—As Engineer of Maintenance of Highways and Sewers in the Borough of Manhattan, the speaker knows that in the process of cleaning the streets after a storm a great deal more than snow is dumped into sewers. That is a point that has to be considered very carefully. After the snow has been on the ground two or three weeks and the Street Cleaning Department begins to clean up, there is a mixture of everything imaginable dumped into the system.

Another feature is that, instead of snow, in many cases, it is ice in large chunks. These may be as large as the manhole, which is 22 in. in diameter, and the ice may be 3 to 6 in. thick.

Half the sewers in Manhattan were built before 1870, and they are in a bad condition. A piece of ice striking some of these sewers will rip out a course of bricks; therefore, the cost of repairing and cleaning sewers has to be considered. The damage that may be done to the sewers is a serious proposition. The theoretical time required for snow to melt, may be calculated; but it is not snow, it is ice, that causes trouble for a large proportion of the time.

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BOUNDARY SURVEYS

Discussion*

By William H. Richards, Jr., M. Am. Soc. C. E.

William H. Richards, Jr.,† M. Am. Soc. C. E. (by letter).‡—Considerable correspondence coming to the attention of the writer testifies that the civil engineer is more generally interested in the questions relating to boundary surveys than would appear to be the case from the space devoted to this field in current engineering publications. Engineering and scientific activities throughout the public domain usually must have, as their basis, a definite and accurate identification of the areas under consideration. In many instances, the original public land surveys—often poorly executed and now, as a rule, to a considerable extent obliterated—do not form a satisfactory basis for the purpose mentioned. For a number of years, re-surveys of such localities have formed a regular and important part of the program of the Cadastral Engineering Service of the General Land Office.

The enactment of the Stock Raising Homestead Act of December 29, 1916 (39 Stat. 862), and the Leasing Act of February 25, 1920 (41 Stat. 437), necessitating, as they do, separate consideration of agricultural surface and mineral sub-surface rights, introduced a number of new questions in cases involving the re-survey of areas where the original official surveys were so grossly defective or obliterated as to render it difficult to identify the legal sub-divisions, the surface rights to which had been disposed of. Such cases involve full consideration of the equities of the various parties in interest. The adjudication of questions of this character is not, of course, a technical or an engineering problem, but preliminary consideration must be given to such matters, in order that the plats of surveys will furnish an adequate basis for final action by the Interior Department. There has recently been completed the re-survey of a large number of townships in Western Colorado,

* Discussion of the paper by C. T. Johnston, M. Am. Soc. C. E., continued from December, 1927, *Proceedings*.

† Cadastral Engr., Gen. Land Office, Washington, D. C.

‡ Received by the Secretary, November 10, 1927.

embracing lands valuable for oil shale, where questions of this character have developed into some of the most complex surveying and platting problems that have ever confronted the Surveying Service of the General Land Office.

Professor Johnston calls attention to the tendency in some States to register land surveyors separately from civil engineers. This procedure may have a tendency to result in a relaxation of requirements as to qualifications to practice cadastral surveying and a consequent lowering of the standards of such work. In registering professional engineers, many States examine applicants in the branch or branches of the profession in which they intend to practice; that is, civil engineers are examined in structural, hydraulic, highway, municipal, or other sub-divisions of the profession. It would be good practice, in the writer's opinion, to include cadastral surveying as one of the sub-divisions just mentioned and to require professional qualifications of the applicants for this branch.

In the organization of the Surveying and Mapping Division, the Society has developed an agency through which its members may be able to study current practice in cadastral surveys as now executed by Federal, State, municipal, county, and private agencies, with a view to the proper co-ordination of their activities, and to the establishment of standards of practice for the different types of cadastral surveys.

Under present conditions, in many States, a cadastral survey affecting the boundaries of valuable public and private property may be executed by a person of little or no technical training, no financial responsibility, without supervision of any public agency, and with inadequate instruments and personnel. As a rule, the requirements in the various States as to monumentation, preparation, and filing of proper plats and records of survey, are inadequate. Here, it would appear, is an opportunity for an important service to the public by the Society, and the Division of Surveying and Mapping now provides the means through which it may work for the establishment of proper standards of practice in cadastral surveys and for the requirement of professional qualifications for all engineers and surveyors engaged in such activities.

Riparian Boundaries.—The subject of riparian boundaries is, from its very nature, a complex one, and it has raised problems that have perplexed others as they have the author. It would appear to be his opinion that lands adjacent to streams, lakes, or other bodies of water, should have been disposed of with a definite reservation of the shore and the riparian rights thereon to the Government for administration in the public interest. Discussion of such a plan is, of course, somewhat academic, since title to lands adjacent to most bodies of waters throughout the United States is now in private ownership, and the questions as to the extent of the title of the riparian proprietors are no longer subject to public control. It may be of interest, however, to examine the origin of these questions and to review the general principles governing the subject.

Farnham, in discussing the matter, states* that in grants made by the early English kings, which undoubtedly established the common law on the

* "Waters and Water Rights," Farnham, Vol. II.

subject, no attempt was made to retain title to the land under the water. As to lands under non-tidal waters, this practice has continued, and the common law rule is that the grant bounded by a stream will carry title to the point of contact with the grant on the opposite side of the stream; that is, the middle of the stream. As to grants bordering on the sea or estuaries or arms of the sea, the boundaries originally extended to the low-water mark.

These rules have since been modified, and in the United States they are quite different.

The extent of grants, by the United States, of its public lands bordering on streams and lakes, made without reservation or restriction, is dependent on the law of the State in which the land lies. It is well established that a State, upon its admission into the Union, immediately becomes vested, by virtue of sovereignty, with the title to the lands under the navigable waters up to mean high-water mark. In the survey of lands bordering navigable waters, it has been the general policy of the Government to extend the lines of survey only to the high-water mark. (Hardin *v.* Jordan, 140 U. S., 371; Barney *v.* Keokuk, 94 U. S., 338.) The practice as to non-navigable bodies of water has been well stated (Rust-Owen Lumber Company, 50 L. D., 678 (Syllabus)*) as follows:

"With respect to public lands bordering on non-navigable bodies of water, the Government assumes the position of a private owner, and when it parts with its title to those lands, without reservation or restriction, the extent of the title of the patentee to the lands under water is governed by the laws of the State within which the lands are situated."

All navigable bodies of water and other rivers and streams of sufficient size and importance to be segregated from the public lands are meandered at mean high-water elevation. Meander lines, of course, are not boundaries defining the limit of ownership of lands bordering on bodies of water, but are run to ascertain the acreage of land in the fractional sub-divisions. The actual bank or shore of the stream at mean high-water elevation is the boundary; and when, by natural causes, the shore line changes, as a general rule, ownership of the adjoining sub-divisions changes with it. (Railroad Company *v.* Schurmeir, 7 Wall., 286; Mitchell *v.* Smale, 140 U. S., 406.)

In considering the distinction between navigable and non-navigable waters, it is interesting to note that the United States Supreme Court, in a decision dated February 1, 1926 (The United States of America, Appellant, *v.* Holt State Bank, *et al.*), stated the rule as to navigability as recognized and applied in the Federal Courts, as follows:

"The rule long since approved by this Court in applying the Constitution and laws of the United States is that streams or lakes which are navigable in fact must be regarded as navigable in law; that they are navigable in fact when they are used, or are susceptible of being used, in their natural and ordinary condition as highways for commerce, over which trade and travel are or may be conducted in the customary modes of trade and travel on water; and further that navigability does not depend on the particular mode in which such use is or may be had—whether by steamboats, sailing vessels, or flatboats—nor

* "Land Decisions of the Interior Department."

on an absence of occasional difficulties in navigation, but on the fact, if it be a fact, that the stream in its natural and ordinary condition affords a channel for useful commerce."

It is evident, then, from the foregoing discussion that, subject to the Federal control over navigation on interstate streams and bodies of water, the States have jurisdiction over questions relating to riparian boundaries, and the engineer who is dealing with problems of drainage, irrigation, and water-power development, must look to them for guidance as to the extent of ownership of riparian proprietors in lands under the waters.

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AERIAL TRAMWAYS

Discussion*

By Robert C. Strachan, M. Am. Soc. C. E.

Robert C. Strachan,† M. Am. Soc. C. E. (by letter).‡—Without questioning the correctness of the author's formulas relating to unloaded spans, it appears that the inherent complexity of the equations, when the origin is taken at one support, might well be avoided; more especially since the operations in problems of the catenary, even under the most favorable assumptions, are as a rule not simple.

Let the origin for the curve shown in Fig. 9§ be taken at the lower extremity of the parameter as indicated in Fig. 23.

Use the author's notation and, in addition, let z = parameter $= \frac{t}{w}$; $\frac{S_1}{z} = u_1$; and $\frac{S_2}{z} = u_2$.

Therefore,

$$\frac{S}{z} = u_1 + u_2 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (109)$$

The equation of the curve then takes its simplest form,

$$y = \frac{t}{w}\left(\cosh \frac{x\,w}{t}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (110)$$

or,

$$y = z \cosh \frac{x}{z} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (111)$$

* This discussion (of the paper by F. C. Carstarphen, M. Am. Soc. C. E., published in November, 1927, *Proceedings*, and presented at the meeting of November 2, 1927), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Brooklyn, N. Y.

‡ Received by the Secretary, November 18, 1927.

§ *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2136.

For any point of the catenary distant x to the left or right of the Y-axis, the vertical distance below L or R is,

$$v_L = z\left(\cosh u_1 - \cosh \frac{x}{z}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (112)$$

$$v_R = z\left(\cosh u_2 - \cosh \frac{x}{z}\right) \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (113)$$

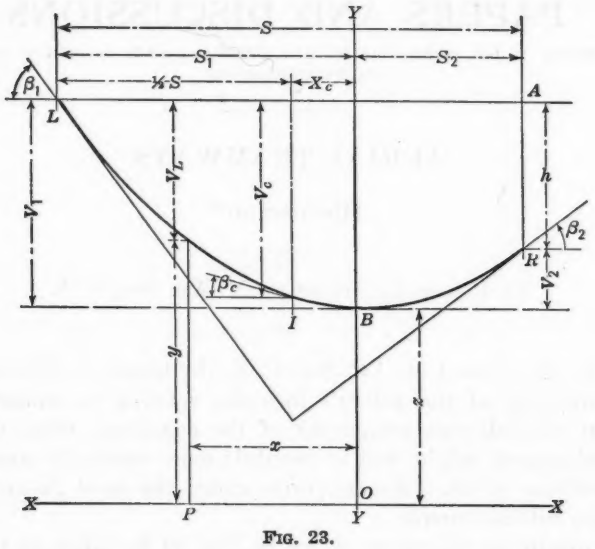

FIG. 23.

The derivation of u_1, u_2, and other required elements, is as follows: It may be shown that,

$$\frac{\frac{1}{2}(u_1 + u_2)}{\sinh \frac{1}{2}(u_1 + u_2)} = \frac{S}{\sqrt{l^2 - h^2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (114)$$

and,

$$\tanh \frac{1}{2}(u_1 - u_2) = \frac{h}{l} \quad \ldots\ldots\ldots\ldots\ldots\ldots (115)$$

From the author's assumed data,

$$\frac{t}{w} = z = 2\,777.7778 \text{ ft.}$$

$$\frac{S}{2z} = \frac{1}{2}(u_1 + u_2) = 0.18000 \text{ hyps*}$$

therefore,

$$\sinh \frac{1}{2}(u_1 + u_2) = 0.18098$$

* A hyperbolic sectorial ratio to designate an angle expressed in radians.

and from Equation (114),

$$l^2 = \frac{(0.18098)^2 S^2}{(0.18)^2} + h^2 \quad \ldots\ldots (116)$$

Therefore,

$$l = 1\,010.405 \text{ ft.}$$

From Equation (115),

$$\tanh \frac{1}{2}(u_1 - u_2) = \frac{100}{1\,010.405} = 0.09897$$

therefore,

$$\frac{1}{2}(u_1 - u_2) = 0.09929 \text{ hyps.}$$

$$u_1 = 0.18000 + 0.09929 = 0.27929 \text{ hyps.}$$

$$u_2 = 0.18000 - 0.09929 = 0.08071 \text{ hyps.}$$

$$\beta_1 = \text{gd. } u_1{}^* = 15^\circ\ 47.9'$$

$$\beta_1 = \text{gd. } u_2 = 4^\circ\ 37.2'$$

No calculation other than interpolation in the hyperbolic tables, is necessary to obtain β_1 and β_2.

$$S_1 = z\,u_1 = 775.806 \text{ ft.}$$

$$S_2 = S - S_1 = z\,u_2 = 224.194 \text{ ft.}$$

$$v_1 = z(\cosh u_1 - 1) = 2\,777.778 \times 0.03926 = 109.055 \text{ ft.}$$

$$v_2 = v_1 - h = z(\cosh u_2 - 1) = 2\,777.778 \times 0.00326 = 9.055 \text{ ft.}$$

If l_1 and l_2, the arcs, respectively, under S_1 and S_2 are required,

$$l_1 = z \sinh u_1 \quad \ldots\ldots (117)$$

$$l_2 = l - l_1 = z \sinh u_2 \quad \ldots\ldots (118)$$

Let x_c = distance from Y-axis to the center of Span $S = \frac{1}{2}(S_1 - S_2)$, and, let $u_c = \frac{x_c}{z}$. Then,

$$u_c = \frac{1}{2}(u_1 - u_2) \quad \ldots\ldots (119)$$

Let v_c = vertical distance from center of span, S, to the curve, then,

$$v_c = z(\cosh u_1 - \cosh u_c) \quad \ldots\ldots (120)$$

and β_c = the inclination of the tangent = gd. u_c

$$x_c = S_1 - \frac{1}{2}S = 275.806 \text{ ft.}$$

$$u_c = \frac{1}{2}(u_1 - u_2) = 0.09929 \text{ hyps.}$$

therefore,

$$\cosh u_c = 1.00493$$

and, by interpolating as before,

$$\beta_c = \text{gd. } u_c = 5^\circ\ 40.8'$$

*This equation states that the angle. β_1 (usually in degrees) is the "Gudermannian angle" corresponding to the hyperbolic sectorial ratio, u.

From Equation (120),

$$v_c = z\ (1.03926 - 1.00493) = 95.361 \text{ ft.}$$

v_c is designated y_c in Fig. 9 and the result of the calculation is given as 95.225 ft.*

The introduction of the author's nomographic charts, Figs. 13,† 14,‡ and 15,§ draws attention anew to the saving in time and labor made possible by this means, with no sacrifice of needful precision. This is an advantage to which reference was made by the writer in his paper‖ on the designing of such charts.

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2135.

† *Loc cit.*, p. 2148.

‡ *Loc. cit.*, p. 2149.

§ *Loc. cit.*, p. 2150.

‖ "Nomographic Solutions for Formulas of Various Types," *Transactions*, Am. Soc. C. E., Vol. LXXVIII (1915), p. 1359.

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BAFFLE-PIER EXPERIMENTS ON MODELS OF PIT RIVER DAMS

Discussion*

By Edward C. Sherman, M. Am. Soc. C. E.

Edward C. Sherman,† M. Am. Soc. C. E. (by letter).‡—It is fortunate that such a complete series of experiments was possible on the models of Pit River dams. The writer concurs in the opinion that there is a question as to the flow over a model being entirely comparable with the corresponding flow over the full-sized structure. However, there can be no doubt that useful information may be obtained from such investigations.

From 1909 to 1911, the writer was Designing Engineer on the Panama Canal, and had charge of the preparation of plans for the spillway dams at Gatun and Miraflores. The larger is at Gatun, and for that location, consideration was given to two types: One with a straight crest and stepped section; the other curved in plan and of ogee section. Models of both were made, primarily to aid in determining the better type to use, but incidentally the opportunity afforded was utilized to test the action of baffle piers in checking the velocity at the toe of the dam.

The lake formed by the great earth dam at Gatun, with its surface 87 ft. above sea level, has an area of 168 sq. miles, and receives the drainage from a water-shed of 1 320 sq. miles. The heavy precipitation of the rainy season frequently causes a high rate of run-off, and as records had been kept for many years, it was possible to determine that the probable maximum momentary rate of discharge at Gatun would be 183 000 sec-ft. The area of the lake is so great, however, that spillway capacity of this amount is not necessary, and the design was made to provide a discharge of 140 000 sec-ft. with the lake at its normal level of 85 ft. About midway in the length of Gatun

* This discussion (of the paper by I. C. Steele and R. H. Monroe, Members, Am. Soc. C. E., published in November, 1927, *Proceedings*, but not presented at any meeting of the Society), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Project Mgr., Bureau of Yards and Docks, U. S. Navy Dept., Washington, D. C.

‡ Received by the Secretary, November 29, 1927.

Dam, there was a rock outcrop which afforded a suitable foundation for the spillway. The extent of this foundation determined the possible length of the crest and, consequently, its elevation. The total distance between abutments is 760 ft., giving a clear crest length of 630 ft., which is divided into fourteen 45-ft. openings. The canal locks were designed on the assumption that the summit level in Gatun Lake would not rise above Elevation 87, and to secure the required capacity without allowing the lake to rise higher, the spillway crest was placed at Elevation 69, with gates of the Stoney type for control.

Tests made with the model of the straight crest dam (Fig. 28) indicated that, with all gates open, the stepped section was effective in destroying the energy of the fall and that uniform flow could be expected below the standing wave at the toe of the dam if the down-stream edge of each step were provided with a strip which would form a pocket or pool. Gatun Lake is frequently regulated by opening a few gates, or only one. It rarely requires full spillway capacity. A test of the model, with only one gate open, gave unexpected results. It was noted that the stream, after striking the first step, turned upward to approximately lake level, without apparent loss of head, and shot forward with sufficient velocity to miss the other steps; so that most of the water fell directly on the floor at the toe of the dam without any check to its force. Similar action on a dam of full size would quickly destroy it; therefore, the results of the test influenced the decision to adopt a dam of ogee section, the model of which is shown in Fig. 29.

The crest of the dam is of a form which facilitates the flow while providing seats for the Stoney gates and the caisson required to permit repairs. The upper part of the down-stream face is parabolic, and shaped so that the nappe adheres to the masonry, and the lower part of the face is a circular curve of long radius. The general plan and sections of the spillway dam are shown in Fig. 32.

The floor of the spillway channel below the dam is 10 ft. above sea level, and the overflowing water, having fallen 75 ft., reaches it with a velocity of about 60 ft. per sec. Some stilling devices other than the effect of the convergence due to the curved crest appeared to be necessary to destroy the energy created by the fall, and a system of baffles was laid out in the floor immediately below the toe of the ogee.

While the model was available, it was possible to make tests to determine whether the proposed arrangement of baffles might be expected to accomplish the desired result. The model was on a scale of $\frac{3}{8}$ in. to 1 ft., the dam being made of concrete placed between wooden baffles cut to the proper section. The crest piers were of wood, grooved for wooden gates, so that the discharge could be regulated. The baffles were made of wood and fastened to the floor with screws to permit trials in different positions. A water supply was temporarily available from the drainage of the hydraulic fill then being placed in Gatun Dam, and a by-pass permitted the regulation of the flow so as to obtain any desired depth on the crest of the model dam.

The time allowed for experiments was limited, because the water was very muddy, and the small pool behind the model was entirely silted up in

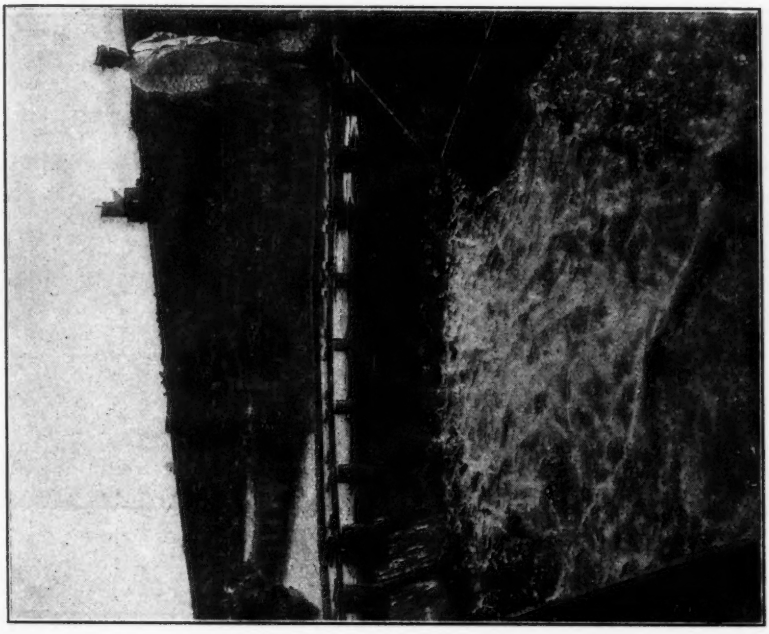

FIG. 29.—MODEL DAM WITH OGEE SECTION.

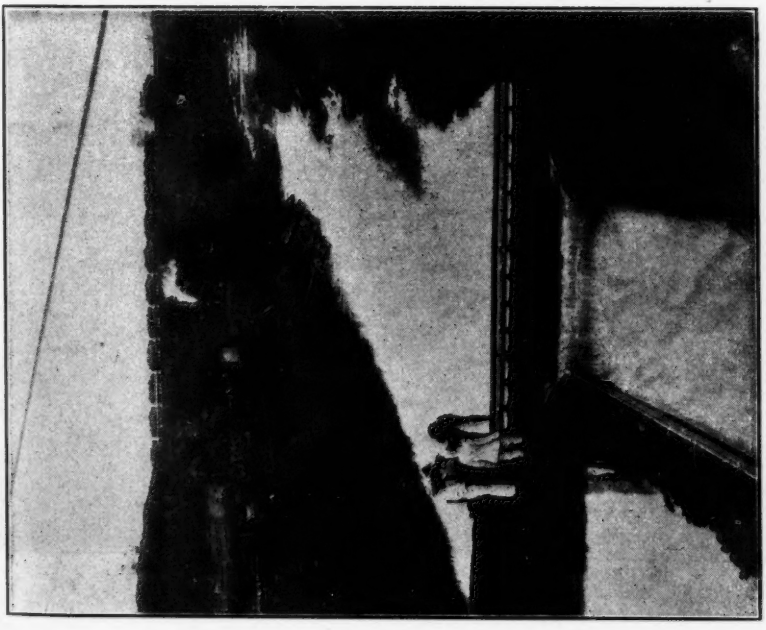

FIG. 28.—MODEL OF STRAIGHT CREST DAM.

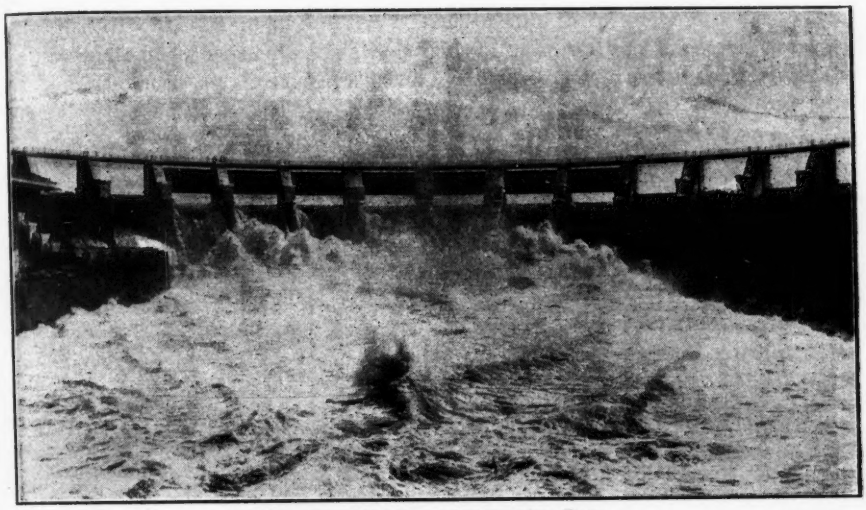

FIG. 30.—VIEW OF COMPLETED DAM.

FIG. 31.—ARRANGEMENT OF BAFFLES, GATUN SPILLWAY DAM.

a few days. Tests were made, however, first, with all contemplated baffles; second, with only the up-stream row of baffles; and, third, without baffles. Only approximate measurements were recorded, but the stilling effect of two rows of baffles, placed as shown in Fig. 32, was obviously greater than with any

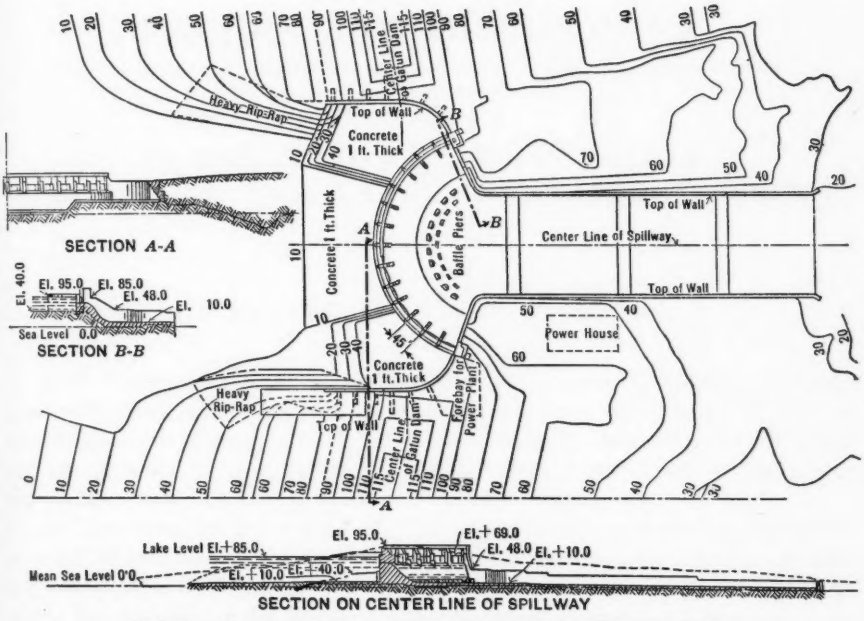

FIG. 32.—GENERAL PLAN OF GATUN SPILLWAY ON PANAMA CANAL.

other arrangement that was tried. The effect of the baffles in establishing a standing wave and uniform flow in the channel is shown in Fig. 29, and a view of the completed dam in Fig. 30. The arrangement of the baffles in the actual structure is shown in Fig. 31.

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PAPERS AND DISCUSSIONS

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RE-DESIGNING CATAWBA STATION FOR SERVICE ON A LARGE TRANSMISSION SYSTEM

Discussion*

BY MESSRS. H. V. JOSLIN AND T. H. HOGG.

H. V. JOSLIN,† M. AM. SOC. C. E.—There is no question but that terracing the arable land would be of advantage to the power companies in that it would diminish silting in their reservoirs; but the speaker does not think missionary work of this nature among the farmers by agents of the power companies would pay for itself for some time to come.

Mr. Lee mentions‡ the flood of 1916, when the river discharge was twice as much as had been heretofore recorded. This flood was considerably greater per square mile of drainage area on the Catawba than it was on the Pee Dee, where the Blewetts Falls Station of the Carolina Power and Light Company is located. The drainage area at the latter point is about twice that at Catawba, and, after data were received from the Southern Power Company of the discharge at Catawba, preparations were immediately made by the Carolina Power and Light Company to strengthen the Blewetts Falls dam and power house in anticipation of a similar flood.

It was discovered that in the event of such a flood the power house itself would probably float, and it was decided to anchor it to the rock below by drilling several holes about 35 ft. long through the concrete, and grouting reinforcing rods into place. Tripod drills of the largest type were used, and steel was obtained from Norway. Several of the holes had to be drilled through the generator-room floor through which a number of conduits had been placed, presumably according to blueprints that were available. Special precautions were taken to avoid hitting these conduits, but without success, and had it not been for the fact that the plant was idle most of the time, due to lack of water, the job would doubtless have cost considerably more. The water shortage was caused by the filling of the reservoir of the Tallassee Power Company farther up the river.

* Discussion of the paper by W. S. Lee, M. Am. Soc. C. E., continued from December, 1927, *Proceedings*.

† Engr., Carolina Power & Light Co., and Affiliated Cos., Raleigh, N. C.

‡ *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2250.

There has not been any flood in the river since, that has approached that of 1916 in intensity, and, inasmuch as since that time three large storage reservoirs have been built above Blewetts Falls, there will probably be no opportunity to test the efficacy of this anchorage. Mr. Lee's paper is very valuable, particularly at this time when, due to the great increase of efficiency in modern turbines and generators, many of the older hydro-electric plants are becoming obsolescent and sooner or later will require drastic remodeling. Probably not all of them will be rebuilt as completely as was the Catawba Station.

T. H. Hogg,* M. Am. Soc. C. E.—Hydro-electric problems in Ontario, Canada, are somewhat different from those in the region to which this paper applies. High flood flows are not experienced as frequently, and more extensive lake systems tend to equalize the flow. To take a specific example; on one of these river systems the drainage area is between 9 000 and 10 000 sq. miles, and lake systems thereon are so extensive as to permit almost complete regulation over a 5-year low-water period. The total run-off per square mile is less than in the South, amounting to 0.6 to 1.3 cu. ft. per sec. in this system.

There are some features of the paper that are of special interest in connection with the present situation in Ontario, as similar problems of load growth and of re-design and rehabilitation of plants are to be met. This is particularly true of the Niagara System, which covers the southwestern part of the Province and is fed by plants situated at Niagara Falls. The Hydro-Electric Power Commission controls three of the four large developments on the Canadian side of the river there; that is, the Toronto Power Plant (having a capacity of 150 000 h.p.), the Ontario Power Plant (180 000 h.p.). and the Queenston Plant (550 000 h.p.). The first unit of the Queenston Plant came into service in the spring of 1922. Nine units are now installed, and in December, 1926, the plant carried a load of 540 000 h.p.; that is, an increase in load of this same amount had taken place in the five years since the plant went into service. It will be seen from this that the problem of providing for increasing loads is strikingly similar to those outlined by the author.

Problems of design are somewhat different from those of the Catawba Station. The Toronto Power Plant was one of the first built at Niagara Falls, and while the gross head is about 180 ft., the operating head, on account of large draft-tube and tail-race tunnel losses, is only approximately 135 ft. The efficiency of the development, based on the gross head, is less than 50 per cent. At Queenston the over-all efficiency is 91%, based on a 305-ft. head. In other words, Queenston delivers 30 h.p. per sec-ft. of water used, and Toronto Power Plant delivers less than 10 h.p. International agreements fix the quantity of water that may be used to develop power, which means that to obtain additional power it will be necessary to curtail the output of the low-head and low efficiency plants and use the water so released more efficiently. This, within a short time, may compel the shutdown of the Toronto Power Plant, unless re-design and rehabilitation can materially modify its water economy.

* Chf. Hydr. Engr., Hydro-Electric Power Comm. of Ontario, Toronto, Ont., Canada.

Changes in the Toronto Power Plant made recently have reduced operating costs. The thrust-bearings in use formerly were of the oil-pressure type, operating at a pressure of 350 lb. per sq. in. These have been replaced by Kingsbury bearings, resulting in an annual saving of $15 000.

Certain operating records from Queenston may be of interest. This plant now operates on a load factor close to 90% for the whole year. The output amounts to 8 500 000 kw-hr. per day, and the writer believes that in the next two or three years this amount may be increased to 9 000 000 kw-hr.

One of the important operating problems on the Niagara River plants is that of ice control and disposal. The Niagara River is, of course, an open river at all times, but it carries enormous quantities of ice, which appears first about the middle of December and is sometimes present as late as the first of June. Ice flow during April and May, however, is not a menace on account of the high air temperature.

Protective equipment at the plant intakes, such as curtain-walls, overflow weirs, and ice runs, is used, and removes as much of the river ice as possible; but at the Queenston Plant, between the intake and the power house, there is 4½ miles of river section of canal, and 9 miles of excavated section. In addition, there flows into the river section the Welland River, about 45 miles long, which drains the country to the west of the Niagara River. Thus, even if all the Niagara River ice is excluded from the intake, it becomes necessary to cope with the ice formed in the Welland River and the long canal. As a matter of fact, the protective measures at the Queenston intake do exclude practically all the river ice, but at the Toronto Plant a considerable quantity of frazil and slush ice passes directly into the penstocks. The greater part of the ice reaching the head-works at Queenston, that is, the ice formed in the canal, is also of the same type, together with a comparatively small quantity of cake ice. The quantities of ice are so great that if it were not cared for in some way the plants would be shut down in an hour or two. In the first year of operation at Queenston, when the intake works were not complete, large quantities of ice gathered and at times curtailed the plant output appreciably, but since 1925 interruptions to the extent of a single kilowatt have not occurred.

If an attempt were made to by-pass the ice by means of the ice chute, which is provided at the end of the forebay, there would be a considerable abstraction of water that has been drawn through the canal, and a power loss of perhaps 30 000 or 40 000 h.p. The alternative is to allow the ice to pass through the turbines, and this has been done, obviating any curtailment of load. The advantages are so great that in the writer's opinion there is no question as to its being the proper solution.

The same method of disposing of the frazil and slush entering the Toronto Plant is followed there. The racks are removed, as otherwise they would quickly become clogged with ice and shut off the flow to the penstocks.

Too much emphasis can be placed on the necessity of keeping ice out of the plants. With the large machines now available, wheel passages are not readily blocked by the ice flowing in the rivers, and little danger can be involved in the method of operation outlined.

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PAPERS AND DISCUSSIONS

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THE SCIENCE OF FOUNDATIONS—ITS PRESENT AND FUTURE

Discussion*

By Messrs. Charles R. Gow, E. P. Goodrich, and Victor A. Endersby.

Charles R. Gow,† M. Am. Soc. C. E.—The speaker has followed with deep interest and appreciation the several contributions which the author of this valuable paper has made to the science of foundation engineering.

The theories advanced in explanation of the behavior of soils under various conditions of use open up many new and interesting avenues of approach in the solution of foundation problems involving the predetermining of soil behavior under specific applications of loads. Undoubtedly, from the standpoint of definite and accepted standards of practice, the general status of foundation engineering is at present in the most unsatisfactory condition of any branch of the science with which engineers are called upon to deal.

There is no recognized authoritative source of information available to the foundation engineer by means of which he may act with the reasonable assurance that the requirements of his problem will be properly and efficiently met. Reliance must be had, in general, on observed experience with regard to previous construction undertakings in the same locality, and where such histories are lacking the designer must often proceed on a basis of uncertain assumptions, which may or may not ultimately prove to have been justified.

Heretofore, soils have usually been classified according to their visible physical properties and engineers have acted on the supposition that these easily recognized characteristics furnished them with a sufficiently satisfactory guide by which to differentiate between soils and identify those possessing similar properties and behaviors.

The results now made available to the profession as a consequence of the author's research investigations throw much new light on the subject. The conclusions thus far would indicate that the essential factors affecting the

* This discussion (of the paper by Charles Terzaghi, M. Am. Soc. C. E., presented at the meeting of the Structural Division, New York, N. Y., on January 20, 1927, and published in November, 1927, *Proceedings*), is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion.

† Cons. Engr. and Contr., Boston, Mass.

behavior of soils under pressure are rather more obscure and complex in their nature than has hitherto been generally assumed.

Obviously, such factors as voids ratio, permeability, compressibility, and elasticity, cannot be determined by any superficial examination of soil samples. Intricate and somewhat elaborate laboratory experiments are necessary for securing this information. Yet, assuming the validity of the author's reasoning, these elements must be known before it is possible to predict, with any degree of accuracy, the capacity of a given soil to support a specified load.

The most encouraging feature of this paper is the evidence that the subject of soil mechanics is at last being studied in a scientific manner and that, as a consequence, there are now some positive experimental data which serve to explain many well recognized but heretofore little understood phenomena.

It is known, for example, that sands of similar appearance as to shape and size of grain may behave quite differently under equal conditions of loading. Likewise, certain clays appear to be highly compressible, whereas, other soils of identical appearance will sustain considerable loads without appreciable consolidation. Again, the driving of piles into a given soil may result in a pronounced upheaval at the surface level, while another apparently identical set of conditions produces little or no such action.

It is not suggested that the author's analysis presents a practical method of forecasting these variations in soil behavior; but it indicates the direction of inquiry that should be followed in order to discover a satisfactory answer for apparent inconsistencies.

It is true that a determination of the essential factors that would make soil identification positive would probably involve more elaborate and expensive procedure than the ordinary engineer could be expected to command. Nevertheless, once the correctness of the author's theories have been established by a sufficient number of laboratory experiments on soils of known behavior, it should be possible, by means of further research, to calibrate these results against more simple rule-of-thumb tests that could be applied in the field. If a satisfactory method can be developed for accomplishing this result, engineers will have made the greatest single step in foundation engineering that has so far been taken.

The author's analysis of the relation between settlement and foundation area* is extremely interesting and more or less in agreement with accepted beliefs; yet it probably will be unwise to assume too readily that such settlements can be reduced to common formulas until at least there has been more practical corroboration of the theory by actual experience.

The author's observations respecting the application of the customary pile-driving formulas† are probably justified by the facts, since there are few engineers, having occasion to drive piles under varying conditions, who have not been impressed by the variation between actual and computed capacities. Fortunately, the so-called *Engineering News* formula has usually given results that are at least on the safe side, and until engineers become more enlightened respecting the factors that produce the observed departures from

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2265.
† *Loc. cit.*, p. 2273.

theory, they will probably do well to stick to that formula as the most dependable guide available.

The author does well to emphasize once again the fallacy of relying too strongly on the results of individual pile tests in determining the average pile load of the cluster.* His analysis of the distribution of soil stresses, as well as his explanation of the variation between dynamic and static resistance, is especially interesting and enlightening.

Of unusual significance is the author's presentation of the compression and rebound curves (Fig. 14)† developed by laboratory experiments on varying types of soil, particularly the comparison presented by similar experiments made on synthetic clays produced by mixing different percentages of powdered mica with fine sand. To those who have been seeking an explanation of the characteristic behavior of clays, as compared with sands, this presentation is especially interesting.

On the whole, it may be said that the author has, by means of this paper, added another important contribution to the clarification of the engineer's understanding respecting many of the most troublesome features of foundation engineering.

E. P. GOODRICH,‡ M. AM. SOC. C. E.—About 1900, the speaker abandoned the use of the *Engineering News* formula. A demonstration of its ineffectiveness was had not long ago in connection with some work that was being supervised by a certain governmental group. Certain requirements were laid down as to the supporting power of the piles, and the *Engineering News* formula was to be used. Those responsible were asked if they would be satisfied with a certain weight of hammer, dropping a certain number of feet. They said they would. The speaker then told them to apply the formula and they found that the result was a negative quantity.

Some data with reference to one of the questions that Professor Terzaghi has propounded, that is, a test of the distribution of stress under a square foundation under which the soil was rather cohesive, are as follows: Some years ago the speaker took a timber platform 4 ft. square, penetrated it by plungers located along a diameter and a diagonal and found that the diagonal stresses were larger than the diametrical ones. In other words, the corners of the square foundation supported more in proportion than any other part, either the center or the sides.

VICTOR A. ENDERSBY,§ ASSOC. M. AM. SOC. C. E. (by letter).‖—The profession should feel indebted to Professor Terzaghi for the clear line he draws between soils to which the *Engineering News* formula is applicable, and those to which it is not. Among field engineers directly in charge of pile-driving, there are a great number who regard this formula as applicable, without modification or question, to every circumstance. There is an equal number who —perhaps more logically—refuse to believe in any formula whatsoever, and

* *Proceedings*, Am. Soc. C. E., November, 1927, Papers and Discussions, p. 2281.
† *Loc. cit.*, p. 2290.
‡ Cons. Engr., New York, N. Y.
§ Constr. Engr., State Highways, Los Angeles, Calif.
‖ Received by the Secretary, November 28, 1927.

who regulate their pile-driving by guess, by the penetration they can force a contractor to attempt, or by the approach of five o'clock. Every practical engineer finds, sooner or later, that there is a great difference in the reliability of formulas in different soils, but there are not many who have any very clear idea as to general distinctions.

It seems very true that the most that can be hoped in regard to the science of foundations is that it become a reasonably reliable empirical system. One is especially impressed with this after such an experience as the writer had on one structure. Certain piles came to refusal in sand at a penetration which he considered unsatisfactory. A jet not being available at the time, he acted on a highly irrational "hunch" and suggested that the piles be left over night and an attempt be made to drive them next day. When this was done, the piles continued to drive with surprising ease, application of the *Engineering News* formula showing that as they stood they had lost 40% of their theoretical bearing power in a few hours. The excavation in which they were being driven was at the time 25 ft. below sea level, and 27 ft. below permanent water line, so that there could have been no question of a change in the texture of material due to subterranean water flows. There was evidently a decompression of some kind, but the occurrence is still a mystery to the writer, which he would like very much to have definitely explained. According to Professor Terzaghi's thesis, this might occur in some soils, but not in sand. The writer has heard a rumor of a similar occurrance on the Gila River in Arizona.

He has had occasion also to test the driving in adjacent and uniform material under three different hammers, a Union No. 2, double-acting, and Vulcan Nos. 1 and 2, single-acting. The theoretical bearing power for piles of uniform size at uniform penetration with these three hammers was practically the same; the result was a good test for consistency of the formula as applied to various types of hammers.

MEMOIRS OF DECEASED MEMBERS

NOTE.—Memoirs will be reproduced in the volumes of *Transactions*. Any information which will amplify the records as here printed, or correct any errors, should be forwarded to the Secretary prior to the final publication.

JAMES EKIN ALLISON, M. Am. Soc. C. E.*

DIED SEPTEMBER 19, 1926.

James Ekin Allison was born at Xenia, Ohio, on May 10, 1865. He was the son of Matthew Corey Allison, a manufacturer at Xenia, and of Frances (Ekin) Allison. He attended the public schools of Xenia and prepared for Harvard University, at Andover, Mass. He received the degree of Bachelor of Arts at Harvard in 1887.

Mr. Allison's first professional experience was as Manager and Engineer of the Xenia Gas, Light and Coke Company, which position he held for two years. He left this work to become Manager and Chief Engineer of the Southern Manufacturing Company, at Nashville, Tenn. This and miscellaneous enterprises occupied his attention until the Spanish-American War, when he was appointed Captain and Adjutant of John Logan's Volunteer Illinois Cavalry.

From 1898 to 1907 Mr. Allison practiced as a Consulting Engineer in St. Louis, Mo. During this period he was employed by the Louisiana Purchase Exposition and invented the gates which were used at that Exposition, and which automatically recorded, at a central control office, the entrance of each person passing through them. In 1907, he became Head of the Department of Boilers and Elevators and Chairman of the Board of Engineers for St. Louis. He held this latter position until 1909, when he was appointed Commissioner and Chief Engineer of the St. Louis Public Service Commission, in which capacity he served until 1913. From 1913 until his death, he maintained an engineering office in St. Louis, where, as a Consulting Engineer, he specialized in public service valuation work. In this work he became very well known and was called upon by utilities and municipalities in all sections of the United States and Canada. Many of the largest utilities in the country came to him as clients, and among the municipalities were St. Louis, San Antonio, Tex., and New Orleans, La.

Mr. Allison held the principle that reproduction new costs should govern in finding the value of the physical items of a plant. His belief in this principle was founded on a deep and careful study of economics. He was an early and outstanding opponent of the theory of depreciation based on the so-called straight-line method.

He wrote many treatises and articles on valuation and other economic subjects. The most noteworthy of these are "Analysis of Rate Calculations" (1910); "Ethical and Economic Elements in Public Service Valuation" (1912); "Should Public Service Utilities be Depreciated to Obtain Fair Value

* Memoir prepared by the following Committee of the St. Louis Section: Messrs. Edward Flad, *Chairman*, J. H. Porter, C. E. Smith, Members, Am. Soc. C. E. and H. J. Pfeifer, Esq.

in Rate or Valuation Cases" (1912); "Rate of Return" (1914); "Criticism of Theoretical Depreciation" (1915); and "Theoretical Depreciation" (1917).

He held membership in a number of social clubs and learned societies. Among his other technical affiliations the following may be mentioned: Engineer's Club of St. Louis, American Institute of Consulting Engineers, and American Society of Mechanical Engineers. He also was, for several years, a member of the Faculty of Washington University, at St. Louis.

Mr. Allison had an active and keenly analytical mind which delighted in discussions of abstract theories and the effort required to ally them with everyday practical problems. In his valuation work he strove to fasten all his methods to basic economic principles. His efforts were to establish methods whereby values could be ascertained that would be true in all cases, for the future as well as for the present. His forceful and attractive personality made many sincere and enduring friendships.

On October 15, 1895, he was married to Jeanie Bass, of Nashville, Tenn. He is survived by his wife and one child, Frances Allison Porter.

Mr. Allison was elected a Member of the American Society of Civil Engineers on October 1, 1913.

GEORGE DANA EMERSON, M. Am. Soc, C. E.*

DIED AUGUST 23, 1927.

George Dana Emerson was born at Wilmington, Del., on August 20, 1876, the son of Charles and Katherine (Hill) Emerson. His early boyhood was spent in Colorado where he received his preliminary education.

After graduation from the East Denver High School, Mr. Emerson entered the University of Colorado, where he remained for a period of two years before transferring to the Massachusetts Institute of Technology, at Cambridge, Mass., from which he was graduated in the year 1899.

His first employment was with the Edison Electric Illuminating Company, at Boston, in the making of surveys and preparation of plans for underground conduit construction. For a brief period he acted as Assistant to Howard A. Carson, Hon. M. Am. Soc. C. E., Consulting Engineer, in connection with some special studies for sewer systems.

Early in 1900 Mr. Emerson accepted employment with the Massachusetts Telephone and Telegraph Company, assuming charge of the survey and plan work involved in the installation of an underground conduit system for its lines in the City of Boston. Later, he supervised the construction of the works.

During 1901 and 1902 he was engaged by the Union Iron Works at San Francisco, Calif., on designs for various electrical and mechanical appliances.

In February, 1903, Mr. Emerson entered the employ of the Boston Transit Commission as a Special Engineering Assistant. For the first few years he was engaged chiefly in connection with experimental work, drafting, and

* Memoir prepared by Charles R. Gow, M. Am. Soc. C. E.

designs for structures, ventilation, pumping plants, and miscellaneous features. He retained his connection with this service until December, 1917. During this period he filled many important assignments in connection with the development of the Rapid Transit System of the City of Boston.

In 1906 he was given charge of the difficult task of underpinning the Ames Building, at Washington and Court Streets, Boston, preparatory to the construction of Section 7 of the Washington Street Tunnel. This structure, built with walls of solid granite masonry, was at that time the heaviest building in the city. The methods adopted for re-supporting its massive exterior walls in order to permit the locating of the proposed tunnel immediately adjacent to the property line, at a considerable depth below the original footings, involved engineering and construction expedients of unusual ingenuity and novelty.

On the successful completion of this work Mr. Emerson was assigned to supervise the interior finish work of several of the Washington Street Tunnel Stations, and, in addition, the construction of the ventilation system of the Tunnel.

In 1919 he conducted the preliminary studies and prepared structural designs for a tunnel under Beacon Hill, which would extend the Cambridge Subway to a connection with the Boston Subway at Park and Tremont Streets, and, subsequently he assumed charge of this construction. Later, in 1912, when this line was extended to Andrews Square, South Boston, he was assigned to supervise the greater part of the work. The most difficult undertaking involved in this extension was the construction of a sub-aqueous section of approximately 3 000 lin. ft., under Fort Point Channel, which was completed by the pneumatic process.

In the fall of 1917, Mr. Emerson was engaged for a brief period in connection with the building of Hog Island Shipyard. Early in 1918 he was employed by the firm of Fay, Spofford, and Thorndike, Consulting Engineers, in connection with the building of the Boston Army Supply Base where he exercised supervision over all construction work amounting in the aggregate to nearly $25 000 000.

On September 28, 1918, he was commissioned Captain in the United States Army and assigned as Assistant to the Constructing Quartermaster at the Boston Army Supply Base, in which position he served until February 20, 1919.

At the conclusion of his military service, Mr. Emerson retired from the active practice of his profession and made his home at Cocoanut Grove, Fla., devoting himself to fruit-growing and other local interests. For several years he served as a member of the Cocoanut Grove City Council and participated in many activities connected with the welfare and development of that community.

He was extremely fond of outdoor life, and spent considerable time in travel, mountain climbing, horseback riding, canoeing, and other forms of open-air exercise.

He was a member of the Florida Society of Civil Engineers, the Miami City Club, the Cocoanut Grove Community Club, the Appalachian Mountain

Club, the Sierra Club, the Colorado Mountain Club, and the Mountaineers, of Seattle, Wash.

Mr. Emerson possessed a lovable personality. He was by nature somewhat quiet, reserved and conservative in manner, but extremely courteous and considerate in his treatment of others. He was uniformly just, generous, and tactful in the discharge of his responsibilities toward those with whom he had occasion to deal. He was admired alike, by his subordinates and superiors. His unusual sound sense, engineering knowledge, and rare good judgment elicited the highest praise from his associates, and from those whose activities he had occasion to supervise. Few engineers of his period possessed as fine a grasp of the essential principles of construction work. He had the rare ability of obtaining the highest quality of workmanship in all the undertakings assigned to him, without appearing to be unfairly exacting in his demands. His accomplishment in this regard stands as a worthy model, deserving of wide emulation.

He was married on January 7, 1911, at Somerville, Mass., to Mildred A. Avery, who survives him. He died very suddenly at Estes Park, Colo., on August 23, 1927.

Mr. Emerson was elected an Associate Member of the American Society of Civil Engineers on June 3, 1908, and a Member on April 14, 1919.

EDWARD AUGUSTUS WHITE HAMMATT, M. Am. Soc. C. E.*

DIED MARCH 18, 1927.

Edward Augustus White Hammatt, the son of Samuel Parkman and Maria (White) Hammatt, was born in that part of West Cambridge, Mass., that is now Belmont on September 30, 1854, and received his early education in the public schools of that place. Later, he entered the Massachusetts Institute of Technology at Cambridge, Mass., from which he was graduated in the Class of 1875.

After his graduation, Mr. Hammatt was employed as Rodman on a preliminary railroad survey, and, subsequently, as Rodman on the construction of the water-works, at Newton, Mass., until he became Assistant Engineer with the late J. P. Frizell, M. Am. Soc. C. E., in 1876. The following year he was engaged in miscellaneous work, and, in 1878, was appointed Draftsman, on the water-works at Lewiston, Me., and, later, as Assistant Engineer for the New York and New England Railroad. During 1879 he served as Draftsman for the American Tool and Machine Company, and also for the Boston Bridge Works, and, in 1880, he became Assistant Engineer in the office of the Proprietors of Locks and Canals on the Merrimac River, at Lowell, Mass. During the following three years he was employed as Chief Assistant Engineer with the late M. M. Tidd, M. Am. Soc. C. E., on water-works construction at

* Memoir prepared by George Bowers and E. S. Dorr, Members, Am. Soc. C. E.

Marlboro, Hudson, Weymouth, Holbrook, Randolph, and Hyde Park, Mass., and at Gardiner, Me.

In 1883, Mr. Hammatt opened an office in Boston, Mass., where he engaged in private practice as a Civil Engineer. In addition to the regular work coming to such an office, of which he took charge, he designed and built several sewerage systems, and did considerable topographical and hydrographical work for power development projects. In 1891, he opened a branch office in Hyde Park, where he was engaged in the engineering work of the city.

In 1906, he went with Charles T. Main, M. Am. Soc. C. E., and a party of engineers on an expedition to the Arros River, in Mexico. Their object was to investigate a hydro-electric development project for furnishing electric current to mining camps in Northern Mexico and in Arizona, and to select a site for a dam, head-house, and similar structures. Mr. Hammatt was left in charge, and remained there for about a month, doing what he termed later, the hardest piece of work he ever attempted. He found numerous dam sites with heads varying from 70 to 285 ft. Owing, however, to lack of provisions, time, and the great expense entailed, the project was temporarily abandoned, and Mr. Hammatt returned to Hyde Park.

In 1912 he removed to Newton Center, and, in 1916, to South Orleans, Mass., where he continued his private practice as Civil and Hydraulic Engineer until his death.

He was married on October 16, 1882, to Mary Louise Fifield, who, with four children, Richard Fox, Francis Parkman, Robert Wallace, and Alice Louise, survives him.

Mr. Hammatt had served as Secretary-Treasurer and Historian of the Class of 1875 of the Massachusetts Institute of Technology for forty-two years. He was a member of the Boston Society of Civil Engineers, the New England Water Works Association, and the Masonic Fraternity.

He was one of the kindest of men, a friend on whom one could always rely, unobtrusive in character, his life was one of faithful service. As an engineer he ranked high, loving his profession and serving it to the last.

Mr. Hammatt was elected a Member of the American Society of Civil Engineers on June 5, 1901.

ALEXANDER MAITLAND, JR., M. Am. Soc. C. E.*

DIED DECEMBER 24, 1926.

Alexander Maitland, Jr., was born on February 9, 1866, on a farm, near Richmond, Mo. He was the son of Alexander Maitland and Mary Grieves (Oliphant) Maitland and the eldest of a family of eight children, two sons and six daughters. He was of Scotch-Irish descent, his father's parents having

* Memoir prepared by George F. Maitland, Herbert S. Crocker, and Milo S. Ketchum, Members, Am. Soc. C. E.

been born in Scotland, and his mother's parents having come from Ireland. On his father's side, he was a descendant of the Scotch Earl of Lauderdale.

He attended a country school until 1886, when he entered the University of Missouri, from which he was graduated in 1889, with the degree of Civil Engineer.

After his graduation from the University of Missouri, Mr. Maitland entered the service of the King Bridge Company, of Cleveland, Ohio, as a Draftsman, and for a period of two years he was employed by this Company and the Keystone Bridge Company, of Pittsburgh, Pa., in estimating and designing various classes of steel structures. In 1892, he returned to the University of Missouri as Assistant Professor of Civil Engineering.

The following year he entered the employ of the Missouri Pacific Railroad Company, as Assistant Engineer in the Bridge and Building Department, and remained in the service of this Company until 1899. His work consisted of the design and supervision of the construction of substructures and superstructures for permanent bridges and buildings.

From 1899 to 1900 Mr. Maitland was engaged as Resident Engineer at Kansas City, Mo., for the Pittsburgh Bridge Company, of Pittsburgh. From 1900 to 1904 he was Contracting Manager for the American Bridge Company at Butte, Mont., and at Kansas City. From 1904 until his death, he was President of the Kansas City Bridge Company, of Kansas City.

During 1918 and 1919 he was Manager of the Gulf District of the United States Shipping Board, Emergency Fleet Corporation, in charge of the construction of wooden ships, with headquarters at Houston, Tex. For several years, previous to and at the time of, his death, he was, also, a member of the firm of Fuller and Maitland, Consulting Engineers, engaged in the design and supervision of construction of the new Water Plant at Kansas City.

Mr. Maitland was endowed with great mental and physical energy, which made him a leader in civic and social matters, as well as in his professional activities. He was a man of winning personality, who made friends easily and held them by loyalty, and a sense of fair play. He was intensely interested in his life work, and was always willing to contribute his time and engineering experience to all his business associates, especially to the younger members of the profession. His loyalty, and friendship will be treasured by his large circle of friends.

On January 8, 1896, Mr. Maitland was married to Desdemona Henderson, of Pacific, Mo. He is survived by his wife and six children, Alexander, 3d, Gladys, Helen, George, Ralph, and John.

Mr. Maitland was a member of the Masonic Lodge, and of the Benevolent and Protective Order of Elks. He was also a member of the Engineers Club of Kansas City and the Kansas City Club.

Mr. Maitland was elected a Member of the American Society of Civil Engineers on November 6, 1901. He served as a Director from January 1, 1925 until his death.

FRANK OLIN MARVIN, M. Am. Soc. C. E.*

DIED FEBRUARY 6, 1915.

Great achievement in engineering is not always evidenced by the magnitude and importance of the buildings, great bridges, or public works that have been designed and built, nor by the economic importance of the industries that have been conceived and developed by an individual. Many engineers have done such outstanding work along these lines of design, construction, invention, and development that their fame will be lasting; and the world has come, through that work, to a full realization of the great part that engineering has had in the development of the present civilization.

Of equally great human value is the work of the man who, in the training of engineers, not only places emphasis on truth, accuracy, and thoroughness, but who by precept and example urges broadness of training and of interest, the cultivation of things that are fine and beautiful, the participation in civic and social activities, and the living of a Christian life.

When such a man comes to an administrative place in any school of engineering, his influence, in character moulding, in the building of good citizens, and the implanting of high ideals of citizenship, of literature, art, music, and religion, along with a sound and thorough professional training is of incalculable benefit to the profession. The engineering of men, the building of engineers, while not spectacular, is of lasting value to the world.

Frank Olin Marvin, Dean of Engineering of the University of Kansas, was such a man. He lived a quiet, beautiful life. He exerted a great influence in many fields of activity. His life work was as an educator and character builder. He influenced the career of several hundred engineers who will pass on to other generations the ideals of this teacher of the truth.

Frank Olin Marvin was born at Alfred, N. Y., on May 27, 1852. He was the son of the Rev. James Marvin and Armina (LeSuer) Marvin. He was graduated with science honors from Allegheny College, at Meadville, Pa., in 1871. During his years in college his chief interest was in the sciences, especially botany and geology, but he undoubtedly there laid the foundation for the appreciation of literature and the arts which developed so fully in his later years. In 1874, he received the Degree of Master of Arts from Alegheny College.

In July, 1871, Mr. Marvin entered the service of the Missouri, Kansas, and Texas Railway Company as Engineer on Construction. In 1872 he became Chief Draftsman for the Company, and, later, Chief Clerk in the Chief Engineer's Office. He began teaching in 1875, so that his subsequent engineering work was done either during vacation periods or in a consulting capacity. From 1878 to 1880, he was Chief Engineer for the Great Eastern Irrigation Company and designed and located 100 miles of irrigation canals and ditches with the dams and other essential structures. From 1889 to 1892, or 1893, he was Consulting Engineer for the City of Lawrence, Kans., on the revision of sewerage plans, and on the construction of storm sewers, arch

* Memoir prepared by Henry Earle Riggs, M. Am. Soc. C. E.

culverts, and various minor structures. In 1894, and in later years, he designed heavy retaining walls and a stone warehouse for a milling company; as Engineer for the University of Kansas, he had much to do with grading, heating tunnels, and other improvements on the campus; he made various reports on sewage disposal, street railways, and other public works in cities and for institutions in Kansas; and, with the State Geologist, he made an extended investigation of the building stones of the State.

It was as a teacher that he did his real work. In September, 1875, he began teaching in the Preparatory Department of the University of Kansas. From 1876 to 1878 he was Principal of the Lawrence High School. In September, 1878, he became Assistant Professor of Mathematics and Engineering. In 1883 he was appointed Professor of Civil Engineering, and, in 1891, Dean of the newly created School of Engineering, which position he held until 1913, when he retired on account of ill health, and became Advisory Dean,

Of Professor Marvin's forty years of teaching work, thirty-seven years were devoted to the service of the University of Kansas during the period of its growth and development. The School of Engineering was his creation, and his personality and character were impressed on the graduates in Engineering, from the beginning of their instruction in the institution. He stood firmly for thoroughness of training in the basic sciences, for careful preparation and accuracy of presentation in class work. He was mild of manner, gentle of voice, of unlimited patience, and was never unfair or unjust to the students. He did not believe in doing work for the pupils, but insisted on each one doing his own thinking and solving his own problems.

Professor Marvin was one of the active leaders in the work of learned and technical societies. He was a member of the Kansas Academy of Science, Kansas Water, Gas and Electric Light Association, Kansas Engineering Society, and American Society for Testing Materials. He was a Fellow of the American Association for the Advancement of Science and Vice-President of Section D in 1896. In 1901 he was President of the Society for the Promotion of Engineering Education, having been one of the most active workers during its early years. He was a member of Phi Beta Kappa and of Sigma Xi for which he worked very earnestly, and of which he became National President in 1909 and 1910. Some of his most intimate contacts with his students came through his active interest in the Science Club of the University, an organization which brought the students of the Eighties very close to the human side of the members of the Faculty. He always urged upon his students the need, after graduation, for such contacts with other members of the profession.

Professor Marvin was married twice, first on August 30, 1876, to Elinor G. McKean who died October 1, 1899, and afterward on December 31, 1901, to Josephine B. March, of Lawrence, who survives him, and who since his death has been living in California. During the long years that his first wife was an invalid, his constant care for her, and his consequent foregoing of many social pleasures, won for him the highest regard of his neighbors. Home life meant much to him. He built a substantial, comfortable house and found

great enjoyment in his library and study in which practically all his work was done for many years.

No memoir of Professor Marvin would be complete that did not lay stress on his citizenship and on his intense interest in the fine things of life outside of the professional field.

He was a lover of music. He had a fine and well trained voice and was for a time a member of a well known quartette, but it was as an organist that he is best remembered. For twenty-six years, from 1877 to 1903, he was Organist of Plymouth Congregational Church. He wrote a number of compositions for the organ, which were usually prepared for some special occasion. He also wrote a number of Christmas and Easter carols, composing both the words and music.

He was also a lover of art. He owned a notable collection of etchings, and did excellent work in this line himself. His work was the first of the kind done in the West and his first etched plate is said to have been the earliest made in the State of Kansas.

He was a lover of literature. His library bespoke his taste for the fine things of all ages and all nations.

Professor Marvin was a good citizen, although a modest and unassertive one. His only active participation in civic affairs was to serve as a member of the City Council of Lawrence for a two-year term from 1900 to 1902, but as an adviser and councillor he assisted in straightening out many a tangle. For many years he was the unofficial consultant and adviser to the City Engineer, always regarding such unpaid services as the duty of a good citizen.

That his years of service were recognized is amply proved by the fact that the Engineering Building at Kansas University is named Marvin Hall; by the fact that the fine new organ recently dedicated in Plymouth Church was appropriately named the Marvin Memorial Organ; and by the fact that in the Engineering Library of the University, to which his books on professional subjects were given, there is a bronze portrait bust and tablet to his memory, placed by the Engineering Alumni in memory of a teacher and friend. These things show that he was known, loved, and appreciated. There is no doubt that his gentle, quiet, effective work for the University and the community endeared him to all with whom he came in contact, and commanded the respect of those who did not know him well.

The writer knew Professor Marvin, first as a teacher of ability, who never used a harsh or sarcastic word in his classes; then, as an adviser, who could always be depended upon to consider and weigh carefully every matter before passing judgment upon it; finally, as a friend of his maturer years, who seemed to find real pleasure in going over the work his "boys" were doing. He would offer a suggestion here, or a mild criticism in the form of a question there, always seeking to encourage more artistic design, or more finished construction. He was an unerring judge of the ability of his students. The writer has employed a number of them on Professor Marvin's recommendation, every one of whom made good in a large way, and every one of whom is now a successful engineer and an active worker in the Society.

Professor Marvin wrote no books. His writing was confined to papers and addresses before the various societies of which he was a member. The subjects had largely to do with pedagogical matters, but some of these subjects clearly indicate his interests, namely, "The Cultural Value of an Engineering Education"; "The Artistic Element in Engineering"; and "Why not Teach About Men?" Once, when the writer was discussing with him the subject of work for an advanced degree, Professor Marvin urged that it be along purely literary lines, because he thought every one ought to develop along lines other than those of his professional work. He continually urged that each of his "boys" cultivate some fad or hobby that would get him far from the field of his practice.

Some of his early writing was prophetic of the great change in American engineering and public works, which has recently been taking place. The finer design, the greater regard for the artistic, the abolition of the ugly things, that is so marked in present-day practice, were foretold by him more than thirty years ago.

More than twelve years have passed since Professor Marvin's death on February 6, 1915. To those of his students who knew him outside the classroom his presence is just as real to-day as it was when they sat in his classes or chummed with him in the years following their graduation, and his gentle, kindly personality and his many lovable qualities will always be a real inspiration to them. Judged by his works, the great engineering school he founded and built up, and the scores of fine and broad engineers who gained their inspiration from him, not only in accuracy and thoroughness, but in love of the fine things of life and in ideals of good citizenship, he may indeed be called a great teacher. As the writer, in recent years, has come to know more intimately the problems of the engineering schools and made contacts with the leading present-day teachers of engineering, he has been brought more fully to a realization of the true worth of Frank Olin Marvin.

Professor Marvin was elected a Member of the American Society of Civil Engineers on May 5, 1897.

FRANK WILLIAM PERKINS, M. Am. Soc. C. E.*

DIED MAY 18, 1927.

Frank William Perkins was born in Chicago, Ill., on September 5, 1882. On graduating from Grammar School, he accepted a position in the Map Department of the Underwriter's Mutual Insurance Company. After leaving this Corporation, he was employed by various firms as Draftsman.

When Mr. Perkins was nineteen years old he determined to become an architect and for a series of years his evenings were devoted to study. In 1908 he was examined for his Architect's License at the University of Illinois, where he passed with high honors.

From 1906 to 1908, Mr. Perkins was in charge of the design and erection of new buildings at the Morton Salt Plant, at Hutchinson, Kans. For two

* Memoir prepared by Isidore Cohen, M. Am. Soc. C. E.

years thereafter, he was engaged in the private practice of Architecture and Structural Engineering in Chicago.

In August, 1911, he entered the employ of the International Packing Company, as Architect and Structural Engineer, where he remained until 1913. From 1913 to 1916 he held a similar position with the Morton Salt Company, in charge of the design and construction of building operations at Hutchinson, Kans., Port Huron and Ludington, Mich., and Grand Saline, Tex. At this time Mr. Perkins was regarded as being more experienced in the design, both structural and mechanical, of salt plants than any of his contemporaries. In 1916, he received his engineering license and for the next five years was engaged in private practice, designing domestic, public, and industrial buildings.

In May, 1921, he went to Manistee, Mich., as Designer, Engineer, and Superintendent of Construction of the Ruggles and Rademaker Salt Plant, which had a daily output capacity of 8 000 bbl., and was probably the largest and most modern salt plant in the world.

When Mr. Perkins returned to Chicago in August, 1922, his interest turned to architectural designing, and he was employed as a designer for various architectural firms. From September, 1924, until February, 1927, he was engaged in remodeling the interior of the First National Bank of Chicago, which was incorporated with the First Trust and Savings Bank. From February to May, 1927, he was Chief Designer for Graven and Mayger, Architects, during which time he designed, in part, the Wetzman Theatre in Detroit, Mich., and the Albee and Keith Theatre, in Rochester, N. Y.

It was while working on the latter theatre that Mr. Perkins became ill with spinal meningitis. He died on May 18, 1927. He was married in 1901 to Vera Ramenstein. He is survived by a son, Frank W., and a daughter, Vera.

Mr. Perkins was of a modest and sympathetic disposition, a most lovable and agreeable companion. To those who knew him, his friends and associates, his death was a severe shock. The community at large has suffered by the premature death of a man and worker who mentally combined a rare esthetic taste with not inconsiderable engineering ability.

Mr. Perkins was elected a Member of the American Society of Civil Engineers on July 11, 1921.

JOHN CLARK SPENCER, M. Am. Soc. C. E.*

DIED JULY 6, 1927.

John Clark Spencer was born in Clinton, Iowa, on August 26, 1869. He attended the State University of Iowa, from which he was graduated with the degree of Civil Engineer in 1891.

After leaving college, he entered the employ of the Chicago Bridge and Iron Company, of Chicago, Ill., holding a position in which he detailed, de-

* Memoir prepared by Charles M. Clark, Esq., Cleveland, Ohio.

signed, and estimated highway bridges. This work included the Winona High Bridge over the Mississippi River, at Winona, Minn., and the Vermillion River Bridge, at Danville, Ill.

In 1893, Mr. Spencer became Engineer for the Portland Bridge and Building Company, of Portland, Ore., in which capacity he had charge of all the preliminary work for the South Eleventh Street Viaduct, Tacoma, Wash. Two years later, as Engineer for the Clinton Bridge and Iron Works, Clinton, Iowa, he was in charge of the office and shop work comprising the design and construction of the Mississippi River Bridge, at Brainard, Minn. In 1897, he completed a course in law at the Cleveland Law School, Cleveland, Ohio, and successfully passed the bar examination. He practiced law to some extent, but soon returned to engineering.

From February, 1898, to May, 1900, he was employed with the Keystone Bridge Works, at Pittsburgh, Pa., as Squad Master in charge of bridge, viaduct, and heavy mill building. At this time, he also made preliminary studies for new Bridge Shops, and after being transferred to the Designing Department in 1899, he planned the major part of Hot Metal Bridge for the Union Railroad Company, at Rankin, Pa.

In May, 1900, Mr. Spencer served in the Berlin and New York Offices of the American Bridge Company. As Designing Engineer, he made the plans for all classes of mill and factory buildings, pier sheds, coal bunkers, and similar structures. Subsequently, he was employed for one year with the Brown Hoist and Machinery Company, on the design of coal and storage plants. Three years later, in the employ of the American Bridge Company of Pittsburgh, he designed all classes of structural steel for manufacturing, office, and mercantile buildings. Among these were the Masonic Temple, at Carnegie, Pa.; the Maloney Building, at Pittsburgh (thirteen stories, not built); the Candler Building, at Atlanta, Ga.; and the high, steel frame, power house for the Central Heating Company, at Detroit, Mich.

In 1905, Mr. Spencer opened an office in Cleveland, where he practiced as a Consulting Engineer. He designed the steel work for the Rockefeller Office Building, as well as several other large buildings in Cleveland and in various other cities. His last work was the planning of the steel construction for the new, 22-story, Brotherhood of Locomotive Engineers Bank Building, at Cleveland, which was completed previous to his death.

Mr. Spencer was always unselfishly interested in Cleveland's civic affairs and gave much of his time to the study and investigation of contemplated city improvements. He was a close observer of men, and very active in his profession up to within a short time of his death.

He was a great lover of books, his studies covering the arts, sciences, philosophy, and law. He was very modest and unassuming, and impressed all with whom he came in contact as dependable and honorable—a gentleman in every respect. He was very approachable and had helped many young men to get a start in life, either by good advice or by financial assistance. Those with whom he was associated learned to know him as one inspired by the highest motives.

He was married, soon after his graduation from the Iowa State University, to Mary E. Huston, the daughter of J. M. Huston, County Surveyor and Engineer, of Clinton, Iowa, who survives him. They had known each other since childhood, and their home life was ideal.

Mr. Spencer was elected a Member of the American Society of Civil Engineers on December 6, 1904.

JOHN MILTON GOODELL, Affiliate, Am. Soc. C. E.*

DIED JUNE 21, 1927.

John Milton Goodell, the son of John B. Goodell, was born August 3, 1867, in Worcester, Mass. He was educated in the Public Schools and the Worcester Polytechnic Institute, from which he was graduated in 1888 with the degree of Bachelor of Science in Civil Engineering. He was also awarded the Salisbury Prize for scholarship.

After a year in Europe, studying at Zurich Polytechnic Institute and traveling, Mr. Goodell was engaged as Instructor in Mathematics at Adelphi Academy, Brooklyn, N. Y. Later, he spent a short time in the engineering office of the late Charles A. Allen, M. Am. Soc. C. E., of Worcester.

In June, 1890, he began his career as an editor of technical periodicals by joining the staff of *Engineering News,* of which later he became Associate Editor under the late Arthur M. Wellington, M. Am. Soc. C. E. In 1892, he transferred to the *Engineering Record,* of which Henry C. Meyer, F. Am. Soc. C. E., was Editor and Proprietor. His editorial work, in various connections, continued throughout a great portion of his life, although such work was interrupted by other engagements and received during certain periods but part-time attention. As an Editor, he was able, painstakingly accurate, and sound in his judgment of situations and men.

From 1895 to 1897, Mr. Goodell was Assistant Secretary of the Society and performed valuable service in the establishment of its effective office and publication methods. In 1901 and 1902, he was Resident Engineer on the staff of Joseph H. Wallace, M. Am. Soc. C. E., of New York, N. Y., on operations near Sault Ste. Marie, Mich. From 1913 to 1915, he collaborated with the late Leonard Metcalf, M. Am. Soc. C. E., and Harrison P. Eddy, M. Am. Soc. C. E., in the preparation of their three-volume work on "American Sewerage Practice." In 1915 and 1916 he devoted his energies to the re-organization of the American Concrete Institute, of which Leonard C. Wason, M. Am. Soc. C. E., was then President.

During 1916 and 1917, he did special work for Fairfax Harrison, President of the Southern Railway Company, assisted in completing the work for which the American Highway Association was organized, and was engaged in war-time organization work for several Government bureaus. For the first six months of 1918 he was Employment Manager of the Emergency Fleet Corporation, of New York, N. Y. Subsequently, he was Acting Chairman of the

* Memoir prepared by Alfred D. Flinn, M. Am. Soc. C. E.

National Highway Council, which had charge of all materials, labor, and transportation for street and road work during the last six months of the World War. He also was Consulting Engineer of the United States Bureau of Public Roads, in connection with its re-organization.

The remainder of Mr. Goodell's life was devoted mainly to literary work related to engineering and to statistical investigations, the former including the *Journal* of the American Water Works Association and publicity for the Babcock and Wilcox Company. During his entire life his major interests were in the fields of water-works, highways, and sanitation. He translated Baumeister's "Cleaning and Sewerage of Cities", and was the author of "Water Works for Small Cities and Towns," as well as "Location, Construction, and Maintenance of Roads."

He was a member of the American Society of Mechanical Engineers, American Water Works Association, New England Water Works Association, and the Cosmos Club of Washington, D. C.

Mr. Goodell was elected an Affiliate of the American Society of Civil Engineers on July 29, 1891.